SEWAGE WORKS JOURNAL

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Rational Design of Settling Tanks—Camp

Rockville Centre Activated Sludge Control—Andersen

N. Y. Award for Rockville Centre Report—Emerson—Dappert

Operating Data, Dearborn—Miller

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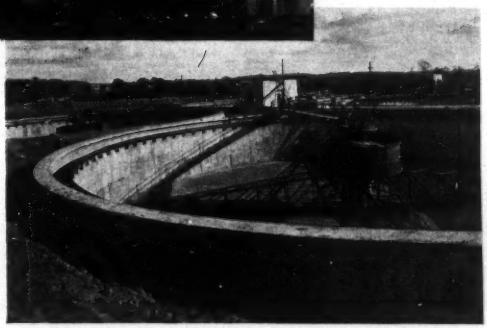

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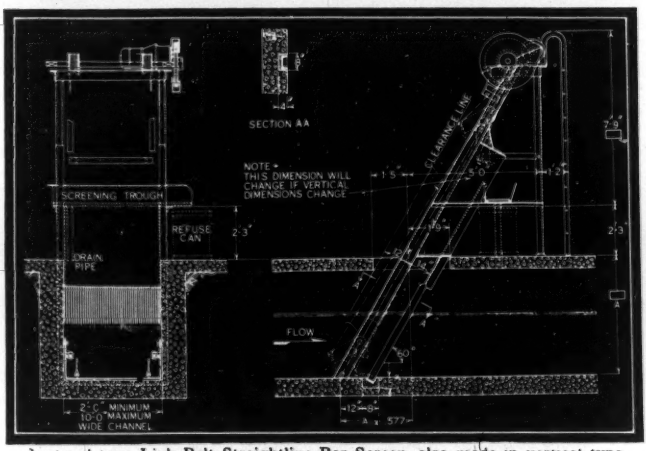

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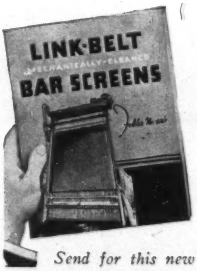

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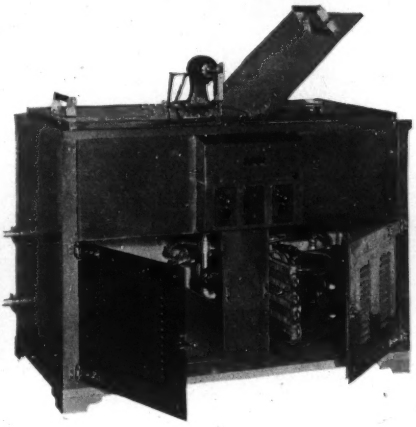

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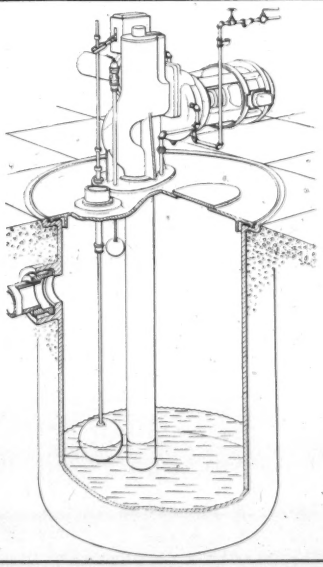

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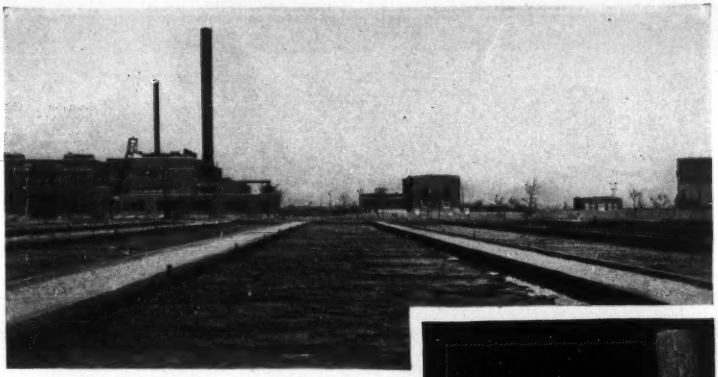

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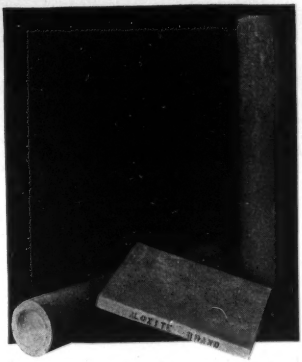

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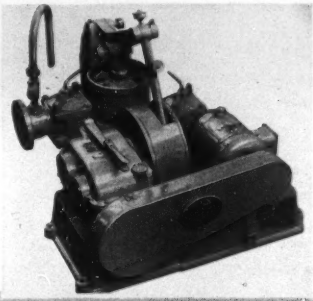

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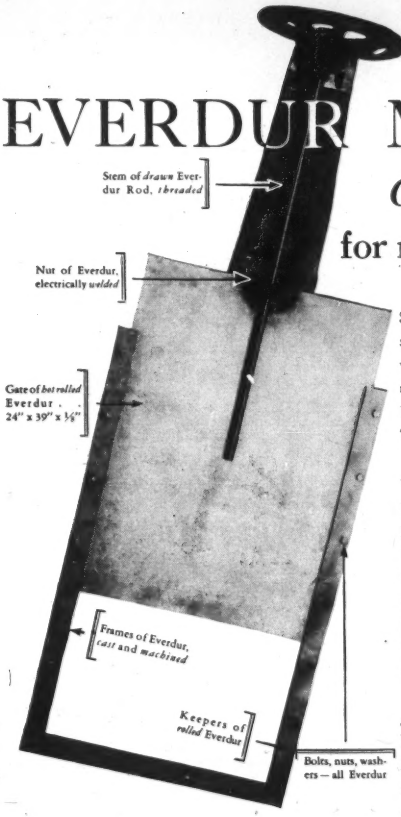
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Sewage Research

OBSERVATIONS ON CERAMIC FILTER MEDIA AND HIGH RATES OF FILTRATION

By Max Levine, Ralph Luebbers, W. E. Galligan and Reese Vaughn

Iowa Engineering Experiment Station, Ames, Iowa

The trickling filter has long held favor as a device for purification of municipal sewage, but recently the introduction of the activated sludge process has threatened to displace the filter, particularly for larger installations. It is probable that the opinion is still held, generally, that the limit of concentration of sewage that can be purified by trickling filters is in the vicinity of about 200 p.p.m. 5-day biochemical oxygen demand, if the rate of application is about 2,000,000 gallons per acre per day, and it is very rare that higher rates of application are employed. A waste of such concentration would, after preliminary treatment in settling tanks, probably reach the filter with a biochemical oxygen demand of 150 p.p.m. and a settled filter effluent of 20 p.p.m. would be considered a very satisfactory efficiency on the part of the filter proper. This is equivalent to a reduction of 86.7 per cent B.O.D. by filtration and secondary sedimentation or a removal of 2,165 pounds of B.O.D. If the filter were six feet deep its capacity for removal of B.O.D. would be approximately 361 pounds per acre-foot per day.

In the course of experiments on the purification of creamery wastes on lath, cinders and other filling materials, it was observed that considerably larger quantities of B.O.D. were effectively removed. It appeared that the biological efficiency of trickling filters, for the removal of oxidizable materials, has been seriously under-estimated. In 1924 it was observed that a skim milk solution with an average oxygen consumed value of 716 p.p.m. when applied to a lath filter at a rate of 2,225,000 gal. per acre per day yielded an effluent of 96 p.p.m. oxygen consumed or a removal of 2,000 pounds of oxygen consuming constituents per acre-foot during the period of operation. (For creamery wastes initial B.O.D. and O.C. values are approximately equal and reductions in B.O.D. by filtration have generally been greater than reductions in O.C.) Cinders were later found to be more efficient than lath but clogging difficulties were encountered.

In 1927–28 experiments on the purification of packing house wastes on trickling filters were in progress at the Decker Packing Company at Mason City, Iowa. An aerated crushed-rock filter, three feet square and ten feet deep, was operated at a rate of 7,000,000 g.p.a.p.d. with a very short (3 minute) dosing cycle. The applied waste had an average B.O.D. of 929 p.p.m., the filter effluent 549 p.p.m. and the effluent after sedimentation 465 p.p.m. The reductions effected were 40.9 per cent

by direct filtration and 49.9 per cent by filtration followed by simple sedimentation. These purifications corresponded to removals of 22,000 and 27,000 pounds of B.O.D. per acre or 2,200 and 2,700 pounds of B.O.D. per acre-foot of filter for direct filtration and filtration coupled with secondary sedimentation, respectively. There was no evidence of clogging in 6 months of operation. It was apparent from these observations that the generally current expectation for trickling filter efficiency, namely the removal of about 360 pounds per acre-foot, was extremely conservative and that at least seven times that quantity could be effectively removed by proper adjustment of the rate of filtration and dosing cycle with the concentrated wastes employed. It remained to be determined whether equally satisfactory results could be obtained with weaker wastes at high rates.

Among the various filling materials employed in the studies on creamery wastes, was a burned clay spiral ring packing. This material consisted of corrugated clay cylinders, 3 in. high and 3 in. in diameter, which were perforated with a 1 in. spiral opening. These units were considered rather large and a high degree of purification was not anticipated but the results were surprisingly good. From the period September, 1926, to January, 1927, B.O.D. reductions of 79.9 per cent were obtained as compared with 97 per cent for gravel run simultaneously. The rate of application was only 550,000 g.p.a.p.d. and the raw waste had a B.O.D. of 513 p.p.m. Nitrification was quite satisfactory on the spiral ring packing, which showed an average nitrate nitrogen content of 6.84 p.p.m. as compared with 13.79 p.p.m. for the gravel. In view of these results with spiral ring packing, it was felt that smaller rings would be much more efficient and still be free from clogging difficulties sometimes encountered with rock filters at low rates of filtration. On the basis of the satisfactory results obtained with the concentrated packing house waste at high rates on the aerated rock filter and the observations with the 3 in. spiral ring packing, experiments on the efficiency of small hollow cylinders known as Raschig rings were begun in 1932. Raschig rings are simple geometric rings or hollow cylinders of equal length and diameter. The wall thickness of the cylinder is only great enough to provide the necessary mechanical stability.

Two sets of observations will be described, the first, which is spoken of as preliminary, consisted of comparisons of 1 in. Raschig rings with the standard crushed rock as filling materials, and the waste consisted of a "synthetic" or artificial mixture of dried sheep manure and powdered skim milk. The second, and more extensive observation, included Raschig rings of various sizes and a special ceramic filling material described later, and the waste consisted of settled domestic sewage.

Preliminary Observations with "Synthetic" or Artificial Wastes

A preliminary experimental plant was built and operated to obtain information concerning the operating characteristics of a ceramic filter

medium. A schematic diagram of this experimental plant is shown in Figure 1. The plant included two experimental filters 2 ft. square in horizontal area and 6 ft. in depth. Galvanized iron tanks supported on angle-iron legs were used as filter cases. One to 3 in. crushed South Dakota quartzite was the medium used in the control filter. This ma-

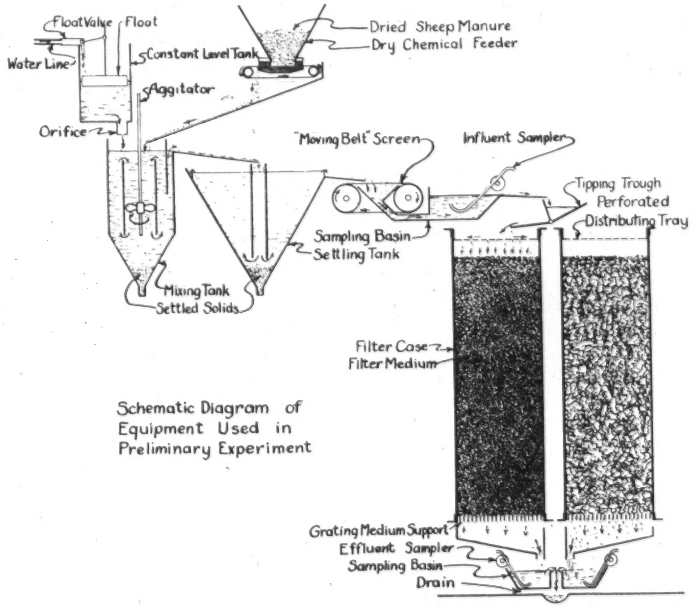


FIG. 1.—Schematic diagram of equipment used in preliminary experiments.

terial is identical with that customarily used in trickling filters throughout the state. One-inch Raschig rings were used as the filter medium in the other filter. The 1 in. Raschig rings were made in the Ceramic Engineering Laboratories under the direction of Professor Paul Cox. Iowa clays were used as the raw material for the rings. The filter media were supported at the bottom of the filter by a wood grid, which had over 50 per cent free air space.

Preparation and Application of Artificial Waste.—The experimental filters were dosed with a synthetic waste produced continuously. The synthetic waste was produced first from dried sheep manure and water and later from a mixture of dried sheep manure and skim milk powder and water. The dried sheep manure or dried sheep manure-milk-powder mixture was fed into a mixing tank by means of a dry chemical feeder of the drag-chain type. Water discharged into the mixing tank from an orifice in the bottom of a constant level tank.

Water from the college water mains was used for suspension and dilution of the synthetic waste. The mixing tank was fitted with a motor-driven propeller in a draft tube. This mixing device prevented the settling of any save the largest gravel particles to the bottom of the mixing tank. The mixed waste was settled for one hour (theoretical time) in a cone-shaped settling tank fitted with a circular weir type of discharge.

The "synthetic" waste was applied to the filters by means of a motor-controlled tipping trough and perforated metal distributing trays. The trays were placed about 6 in. above the filter medium, and sufficiently raised above the filter cases to allow free ventilation from the top of the filters. The effluents from the filters were collected on a sloping tray suspended a few inches below the bottom of the filter cases, the waste discharging from an outlet in this tray.

Sampling.—Motor-operated samplers collected a small sample of the influent and each of the effluents every 5 minutes. A gallon composite sample was collected in this manner in 24 hours. The samplers were spoonlike affairs which dipped into the appropriate sampling basin at 5-min. intervals and removed a small sample, placing it into a suitable container.

Dosing Rate and Cycle.—During the entire operation of this experimental plant the rate of application of the waste was maintained at 2 m.g.p.a.p.d. with a 6 min. dosing cycle. The rate of application was dependent upon the flow of water into the mixing tank. This was maintained constant by maintaining a constant head of water on the orifice by means of a float valve, and by occasional cleaning of the orifice. Since water from the college water main was being used, the orifice did not require cleaning save at rare intervals. The dosing cycle was maintained constant by means of a motor-operated mechanism which permitted the tipping trough to tip only at certain intervals; every 3 minutes in this case. The filters were in this manner dosed alternately, allowing each filter to be dosed once every 6 minutes.

Results with Artificial Wastes

The operation of the plant may be divided into three periods on the basis of the concentration of the applied waste. During the period May 23, 1933, to June 20, 1933, only dried sheep manure was used for waste production. During this period the average B.O.D. concentration of the applied waste was 117 p.p.m. During the second operation period, from June 21 to August 25, a mixture of 1 lb. of milk powder to 16 lb. of dried sheep manure was employed and the average B.O.D. concentration of the applied waste was 567 p.p.m. During the last or third operating period, from August 26 until September 28, a mixture of 1 lb. of milk powder to 8 lb. of dried sheep manure was used and the average B.O.D. concentration of the applied waste was 999 p.p.m.

During the first two operating periods no abnormality of operation was observed on either filter. In the third operating period, however, some clogging occurred in both filters. In the case of the granite filter,

ponding occurred to a depth of several inches over the entire filter. In the case of the Raschig ring filter, ponding was rather superficial, covering only small areas of the surface. In both cases, however, the filters cleared themselves without stopping application of waste or using other remedial measures.

An interesting result was obtained on September 20. At this time the bottom of the filters were closed preventing bottom ventilation, the remainder of the setup being unchanged. The B.O.D. and oxygen consumed reductions immediately became zero. In other words, purification ceased almost immediately when bottom ventilation was stopped.

The results of these three periods of operation are summarized in Table I.

TABLE I.—*Comparison of Granite and Raschig Rings as Sewage Filter Media (Preliminary Experiments)*

Date 1933	Period	B.O.D. (P.p.m.)			% Red B.O.D.		Lb. B.O.D. Removed per Acre-ft.		Relative Stability		NO_3 in P.p.m.		
		Inf.	Effluents								Inf.	Effluents	
			Rings	Granite	Rings	Granite	Rings	Granite	Rings	Granite		Rings	Granite
May 23 to June 30...	1	117	18	21	84.6	82.0	275	266	84	70	—	—	—
June 21 to Aug. 25...	2	567	46	58	91.9	89.8	1450	1410	56	43	2.5	6.06	3.90
Aug. 26 to Sept. 28..	3	999	146	309	85.2	68.8	2080	1830	21	4	4.5	6.66	3.90

The filters operated quite normally, without clogging or other irregularity during the first two periods. The results as shown in Table I indicate only slightly greater purification by the Raschig ring filter than by the granite filter. The reduction of B.O.D. by the Raschig ring filter was not significantly greater than that of the granite filter. Nitrification was distinctly better in the Raschig ring filter. During the third operating period, clogging occurred in both filters several times. In the case of the granite filter the clogging was serious enough to cause ponding to a depth of several inches at times. In the case of the Raschig ring filter no appreciable ponding was observed. In each case, however, the filters cleared themselves without any remedial measure or stoppage of waste application. It appeared that the granite filter was somewhat clogged all through this third period of operation. Much greater purification was obtained with the Raschig ring filter than with the granite filter.

During the third period the B.O.D. of the influent was 999 p.p.m., that of the Raschig ring effluent 146 p.p.m. or a reduction of 85.2 per cent as compared with a B.O.D. of 309 p.p.m. or a reduction of 68.8 per cent on the granite filter. With respect to nitrification, it will be observed from Table I that in the granite filter there was a slight

reduction in nitrate content over that present in the influent or applied waste, whereas in the Raschig-ring filter there was an increase of 2.16 p.p.m. of nitrates. It is apparent from this preliminary observation that the Raschig rings were superior at the high concentration as indicated by the greater reduction of B.O.D. and the better nitrification. The removals of B.O.D. constituents by the Raschig-ring filter were 275 lb, 1450 lb. and 2080 lb. per acre-foot of filter during the preliminary series of observations when the applied wastes had B.O.D.'s of 117, 567 and 999 p.p.m., respectively.

The experiments described below were designed, among other things, to ascertain whether the Raschig rings would show a similar superiority over crushed work when high rates of application were employed.

Observations with Municipal Sewage

The preliminary investigation indicated that Raschig rings have definite merit as filter media. Furthermore, evidence previously referred to warranted the view that filters may be operated at a much higher rate than is customarily used. The use of the synthetic waste employed was found to be impractical for the purpose at hand.

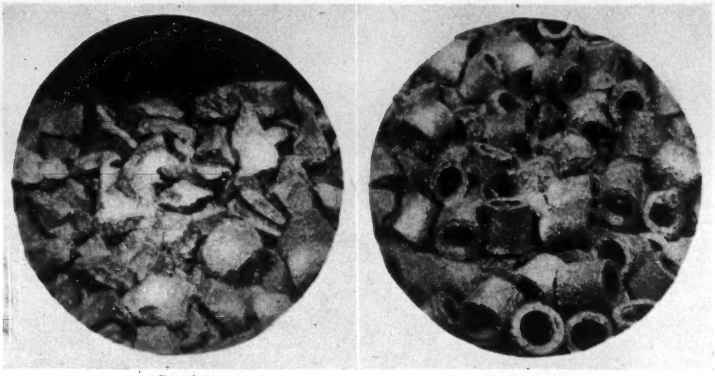
A more detailed investigation was projected employing domestic sewage and having the following objectives:

a. To obtain additional information concerning the feasibility of using ceramic products as trickling filter media.
b. To obtain information concerning the optimum size of such a medium for commercial use.
c. To obtain information concerning the relationship between the physical characteristics of the medium such as surface, free air space, and interstice size, to the purification obtained.
d. To obtain information on the effect of rate of application of waste on purification by trickling filters.

Filter Media.—As in the preliminary investigation, 1 to 3 in. crushed South Dakota quartzite was used in a control or standard filter.

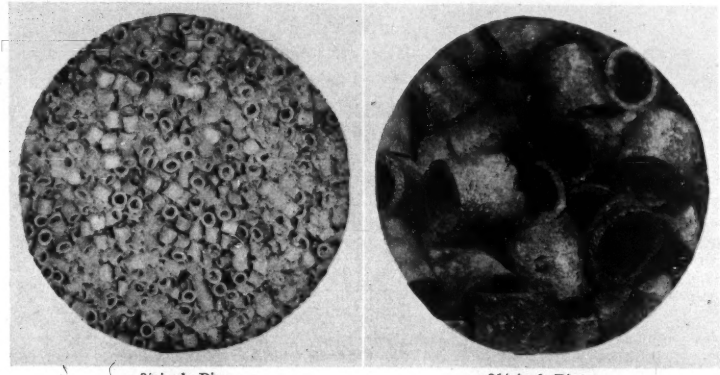
Four sizes of Raschig rings were used in this investigation: ¾ in., 1 in., 1½ in., and 2¼ in. The 1 in. rings used in the preliminary investigation were again used in this investigation. One cubic yard of each of the other three sizes was made in the Ceramic Engineering Laboratory of Iowa State College, under the direction of Professor Paul Cox. Iowa clays from various sections of the state were used as the raw material for these rings. The rings used in the preliminary investigation were not burned very hard. The other three sizes were burned at a somewhat higher temperature.

The ceramic products used in this experiment can be readily produced from almost any of the Iowa clays. No special preparation is required, save for the screening out of pebbles and other foreign material which would clog the die. The rings were made by extruding from an augur machine, followed by hand cutting. In a commercial



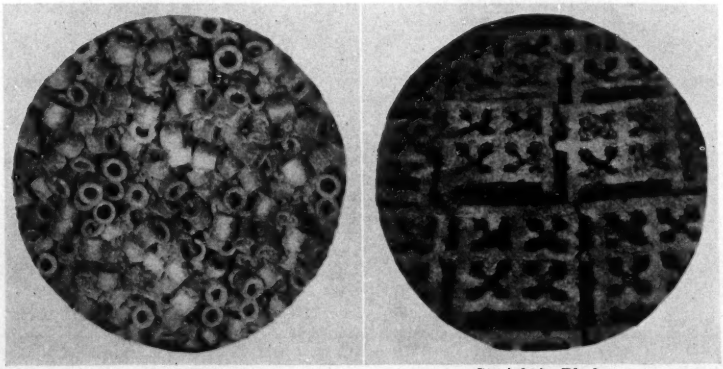
Granite

1½-inch Rings



¾-inch Rings

2¼-inch Rings



1-inch Rings

Straight's Block

FIG 2.—Filter media with microbial films.

plant an automatic cutting device would be used.* Following the cutting operation the rings entered a preliminary dryer for a few minutes. After preliminary drying, the rings were strong enough to withstand considerable handling. A short drying period of an hour or two will prepare the rings for firing. The rings used in this investigation were only partially vitrified. This investigation did not include observation on the freezing and thawing resistance of the rings. It was not felt that this was essential for this study, inasmuch as the rings can readily be burned to any degree of vitrification required to resist freezing and thawing.

A special block was designed and built by Mr. H. R. Straight of the Adel Clay Products Co., after inspecting the preliminary experiment with Raschig rings. This will be referred to as Straight's block, and was used in filter No. 6. This block is a modified building block having four star-shaped vertical channels in each block. The block is roughly 4¼ in. square in a horizontal plane and about 2½ in. in depth. A top view of the block in place in the filter with the microbial film developed is shown in Figure 2. In one filter corn cobs were used

TABLE II.—*Physical Constants of Filter Media*

Material	Length (Inches)	Outside Diameter (Inches)	Inside Diameter (Inches)	Number of Units per Cu. Ft.	Weight Lb. per Cu. Ft.	Surface Area Sq. Ft. per Cu. Ft.	Voids Per Cent	
Granite								
1 to 3 in.						23.4	37	(*a*)
						19.0	46	(*b*)
					90.53	29.98	45.3	(*c*)
Raschig rings								
¾ in.	0.7797	0.7926	0.4845	2925	46.87	75.8	59.36	(*d*)
1 in.	1.025	1.081	0.6346	1174	49.72	52.2	58.14	(*e*)
1½ in.	1.5685	1.549	1.042	340	40.81	35.0	68.21	
2¼ in.	2.341	2.342	1.750	97	30.30	22.7	74.93	
Straight's block								
Theoretical				34.4	56.30	35.8	53.1	(*f*)
Actual				27.1	44.25	28.2	63.1	(*g*)

(*a*) Values given by Butcher. *Industrial Chemist*, **4**: 446.

(*b*) Values given by Zeisberg. *Trans. A.I.Ch.E.*, Pt. II, **12**: 231.

(*c*) Values given by Whisler. Unpublished report, Iowa Eng. Exp. Sta., Ames, Iowa.

(*d*) Dimensions of rings are averages of measurements of 100 rings selected at random. The number and weight of rings per cubic foot were determined by counting and weighing the contents of a box of 1.7288 cu. ft. volume. The surface area and per cent voids were calculated from these measurements, assuming the rings to be cylinders.

(*e*) The one-inch rings were made one year previous to the other sizes and have somewhat heavier walls.

(*f*) The theoretical values were obtained by measurement of blocks, assuming theoretical packing conditions.

(*g*) It was impossible to completely utilize all available space when the blocks were packed in the experimental filter. The values given are those obtained in the filter.

* A suitable cutter has been developed by Mr. John Hug of the Department of Mechanical Engineering, Iowa State College.

but as the purpose and function of that filter was radically different from the other filling materials, the results will not be included in this report.

Physical Constants of Filter Media.—The principal physical constants of the media are given in Table II. The actual dimensions of the Raschig rings are averages of measurements on 100 rings collected at random while placing the rings in the filters. The number per cubic foot was determined by counting the number of rings contained in a box of about 1¾ cu. ft. capacity. The weight per cu. ft. was determined by ascertaining the weight of rings in this measuring box. The surface area per cu. ft. was calculated by considering the rings as hollow cylinders, and taking the area as the sum of the inside and outside areas of the cylinder and the annular ring area on each end. The per cent of voids was arrived at by calculating the volume occupied by the rings, and subtracting this from the total volume.

The surface area of the trap rock contained in a given volume was determined by a graduate student, Mr. Whisler, under the direction of Professor Galligan of the Sanitary Engineering Department. The per cent of voids was calculated from a knowledge of the actual density of the granite and the apparent density in the filter.

The physical constants of Straight's block were calculated on two bases, first considering the block stacked according to theoretical dimensions, as they would be in a large bed, and second as they were stacked in the experimental filter. In the small experimental filter it was necessary to leave some vacant space in each layer. It is theoretically possible to place 34.4 blocks per cu. ft. In the experimental filter only 27.08 blocks were used per cu. ft. The weight per cu. ft. is based on the average weight of a number of blocks. The surface per cu. ft. is calculated from the measured area of one block. Roughness was not taken into consideration in measuring the area of a block. The star-shaped channels were quite rough. As a result of this the surface area given is probably low. The percentage of voids was calculated from the volume of solid material per block and the volume occupied by one block.

The granite and Raschig rings were placed in filters without any attempt at stacking or other arrangement. The media may be said to have been dumped. In the case of the Straight's block, the material was carefully stacked in the filters, alternate courses being offset. There were twenty-five blocks in each course to cover an area of 4 sq. ft.

Filter Construction and Dosing Mechanism.—A schematic diagram of one experimental trickling filter unit is shown in Figure 3. According to this diagram, a motor-driven gear pump lifts the sewage from the Imhoff tank discharge into a distributing trough. This trough is connected with seven weirs over which sewage flows into the dosing tanks for each filter, an overflow siphon to waste sewage, a quick-opening waste or drain valve and the influent sampler. Each of the seven dosing tanks is fitted with a movable partition for varying the ca-

pacity of the tank, and a quick-opening valve "*A*" for discharging the sewage onto a perforated metal distributing tray. The distributing trays are a few inches below their respective dosing valves and about eight inches above the filter media.

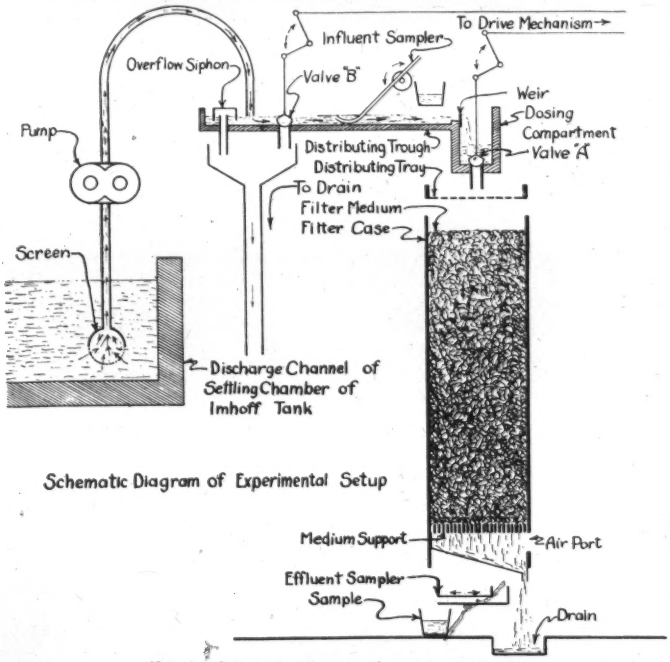


FIG. 3.—Schematic diagram of experimental unit.

The dosing cycle occurs in the following fashion: The pump is discharging continuously into the distributing trough. When the drain valve "*B*" is closed, the sewage flows over the seven small weirs into the seven dosing tanks. As soon as the dosing tanks are filled the liquid level in the distributing trough rises until the overflow siphon is started and carries away the excess waste. At this point the motor-operated drain valve "*B*" opens, and drains the distributing trough and liquid in each of the dosing tanks to the level of the weir in that tank. At the appropriate time the motor-operated dosing valve "*A*" is opened, discharging the contents of the dosing tank onto the distributing tray. The dosing and drain valves are again closed completing one dosing cycle.

The size and dose is determined by the capacity of the dosing tank and is readily maintained constant. The time of the dosing cycle was

maintained constant by means of an adjustable motor-driven mechanism.

The dosing tanks, distributing trough, and filter cases were constructed of thoroughly painted wood. The seven filter cases, each 24 in. on a side, inside dimensions, were built side by side, adjoining filters having one wall in common. The depth of filter medium was six feet. The general features are indicated in Figure 3.

Figure 2 shows the appearance of the various filter media on the surface of the filters after they had been operated for about one year.

Sampling.—Mechanically operated samplers collected small samples from the influent and from each of the six effluents every two minutes. The influent sampler was of a curved copper tube dipper which took samples from the distributor trough. The effluent sampler was of the "snatch" type, so designed as to catch the effluent from each filter for a short time interval, the time interval being fixed so as to obtain the desired quantity of sample. A 2 gal. composite sample was collected in this fashion in 24 hours.

Determinations of B.O.D. and relative stability were made on these 24-hour composites, two or three times a week.

Two determinations of B.O.D. were made, one consisting of the effluent of the filters thoroughly shaken and the other of the supernatant liquid from the Imhoff cones after settling one hour.

Portions of these daily twenty-four hour composites were chloroformed for bi- or tri-weekly composites which were analyzed for various nitrogen determinations, reaction (pH) and oxygen consumed, solids and so forth.

Results

The filters were operated at 2 m.g., 4 m.g., 8 m.g. and 16 m.g. per acre per day. The results for each of these four periods of operation, with different rates of application of sewage, are shown in the tables which follow.

During the first operating period from August 16, 1934, until December 9, 1934, the filters were dosed at the rate of 2 m.g.p.a.p.d. with a dosing cycle of 6 minutes. The plant was operated intermittently for a short period before the first operation period, while run-off rate studies were being made, but no samples were taken during this time. As is to be expected in any experimental setup as intricate as that used in this investigation, mechanical difficulties arose. These were overcome, for the most part, during the early part of the first operating period.

During the second operating period from December 10, 1934, to April 1, 1935, the waste was applied at the rate of 4 m.g.p.a.p.d. with a 3 min. dosing cycle. During this period some irregularity of operation was caused by the extreme cold weather. Gas produced by sludge digestion in the city plant was used to heat the building housing the experimental plant. However, the high moisture content of the sludge gas resulted in condensation, which frequently collected in the gas line and caused stoppage. These interruptions in the gas flow, in part,

resulted in marked variations in room temperature. The extreme cold, together with an inability to get gas and sewage continuously, made it necessary to discontinue the operation of the experimental plant for the period January 20 to February 14. In a large plant the cold weather would not have presented a serious difficulty. The temperature of the

TABLE III.—*Effect of Filling Material on Reduction of B.O.D.*
(2 M.G.P.A.P.D.; 6 Min. Dosing Cycle)

Date 1934	Influent (Settled Ames Sewage)	B.O.D. of Effluents											
		Granite 1 to 3″		Raschig Rings								Straight's Block	
				¾″		1″		1½″		2¼″			
		Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.
Aug. 25	265		27		12		21		21		12		17
31	160		28		23		25		19		52		56
Sept. 1	165		16		41		64		29		11		37
3	102		10		5		6		6		11		9
4	160		24		8		12		12		13		10
5	205		39		11		19		32		42		30
6	120		8		6		12		51		16		61
(g) 6	160		50		5		6		41		20		16
7	166		17		6		—		—		—		—
10	186		47		18		16		35		25		33
11	212		16		5		31*		21		15		17
12	205		27		12		24		30*		29		29
14	130	40	11	18	9	31	13	—	24	25	18	28	4
17	114		6		4		5		7		5		3
20	176	57		22		35*		32*		35*		34*	
22	223	33*		21		32		33*		32*		32*	
24	173	30*		15*		15*		26*		30*		30*	
26	152	48		27		48		64		66		65	
27	250	65		35		43		54		64		70*	
28	218	64		40		52		68		72		72*	
29	185	63		32		45		63		69*		70*	
Oct. 1	145	57		25		38		71*		65		63	
3	135	38		13		21		45		52		52	
4	218	60		33		42		65		65		55	
5	193	39	8	14	6	28	7	59	25	60	27*	49	9
8	135	57	15	19	3	38	6	61	13	60	24	51	12
10	217	50*	32*	14	7	29	12	61	34	65	33	53	27
12	212	45	11	38	5	54	10	60	26	63	15	45	5
15	210	64	30	62	30	32	15	64*	29	65	21	64*	30*
17	230	56	22	50	11	71	30	70*	29	72*	20	67	24
18	230	33	12	29	2	46	8	60	—	58	20	40	15
19	175	53	30	23	11	42	18	67	31	59	26	52	17
22	175	52	17	66	14	58	8	62	19	60	12	62	11
24	192	46	25	47	20	51	12	59	16	58	14	66	15
25	255	51	24	59	25	55	23	56	30	58	19	63	16
26	260	54	31	59	21	54	20	64	25	65	23	65	20
29	227	64	30	71	16	67	26	67	35	67	31	67	27
31	330	66	22	41	7	56	10	67	18	60	20	54	12

TABLE III.—*Continued*

Date 1934	Influent (Settled Ames Sewage)	B.O.D. of Effluents											
		Granite 1 to 3″		Raschig Rings								Straight's Block	
				¾″		1″		1½″		2¼″			
		Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.
Nov. 1	207	65	35	49	16	50	18	67	33	65	30	69	15
5	232	60	23	32	13	61	22	62	21	62	25	62	20
7	177	52	20	33	7	50	14	63	20	65	19	—	8
8	257	66	20	26	4	53	9	59	10	58	10	67	7
9	297	68	36*	46	16	62	27	65	29	74	36	80*	40*
12	175	67	19	32	11	41	8	49	16	72	22	72	14
14	122	44	14	16	6	18	6	49	15	49	25	50	15
19	220	70	16	38	5	40	8	70*	29	68	26	78	22
23	180	76	38	50	15	63	20	72	35	75	33	70	32
28	207	70	36	64	25	67	35	63	30	75	34	67	34
Dec. 5	152	62	32	26	12	45	17	61	26	65	28	67	21
7	140	68	37*	25	9	53	12	71	24	74	26	—	28
No. of analyses	50	37	40	37	40	37	39	36	38	37	39	35	39
Average	186	55.5	24.0	35.4	12.0	45.6	16.8	59.7	24.9	60.7	22.8	58.6	21.2
Average % reduction	—	71.3	87.3	82.6	93.7	77.2	91.2	69.3	86.9	67.9	87.8	70.4	88.4

* Dilution completely depleted. Value shown is that of 100 per cent depletion.
— Analysis not complete.
(g) Grab sample, 2 cycles.

incoming sewage is probably always warm enough to keep the filters from freezing, provided the dosing cycle does not become too long during the low night flow.

During the third operating period from April 2 until June 18, 1935, the waste was applied at the rate of 8 m.g.p.a.p.d. with a 3 min. dosing cycle. The experimental plant operated very uniformly during this period. However, because of spring rains the sewage applied was at times quite dilute. In fact the plant was shut down at the end of the period because of extreme high water which rose into the pump house well, and caused the pumpage of part river water.

After the river subsided the plant was put into operation by Dr. Vaughn at a rate of application of 16 m.g.p.a.p.d. with a 3 min. dosing cycle. This rate was maintained for the period July 3 to Nov. 2, 1935.

For convenience of discussion the most pertinent data will be presented for each of the rates of application of sewage and the results then briefly summarized. In Tables III and III*A* are given the data for B.O.D.'s on the fresh 24-hour composites and chemical analyses of the chloroformed (preserved) samples obtained during the period August 15 to December 10, 1934, when the sewage was applied at the rate of 2 m.g.p.a.p.d. The average temperature of the applied waste was

TABLE IIIA.—*Analyses of Preserved Composite Samples for Nitrogenous Constituents (2 M.G.P.A.P.D.; 6 Min. Dosing Cycle Series)*

Date 1934	Comp. No.	Influent (Settled Ames Sewage)	Effluents					
			Granite 1 to 3"	Raschig Rings				Straight's Block
				¾"	1"	1½"	2¼"	
Nitrate Nitrogen (in p.p.m.)								
Aug. 16–31	I	0.5	4.8	7.2	14.3	2.9	0.4	0
Sept. 1–20	II	0	8.4	22.9	11.7	6.3	7.4	9.4
Sept. 21–Oct. 10	III	tr.	1.7	8.6	15.7	9.0	1.0	3.6
Oct. 11–29	IV	0.5	5.4	15.7	13.2	3.7	0	7.9
Oct. 30–Nov. 17	V	0.6	6.2	26.4	21.5	15.7	6.7	12.8
Nov. 18–Dec. 9	VI	0.8	10.7	30.8	25.1	14.1	5.2	11.0
Average		0.4	6.2	18.6	16.9	8.6	3.4	7.4
Nitrite Nitrogen (in p.p.m.)								
Aug. 16–31	I	0	1.38	1.00	1.74	1.55	3.40	6.70
Sept. 1–20	II	tr.	2.86	2.40	4.40	7.30	3.10	2.60
Sept. 21–Oct. 10	III	0	1.18	1.16	1.46	1.37	1.35	1.74
Oct. 11–29	IV	0	1.22	1.33	1.32	1.31	3.10	3.93
Oct. 30–Nov. 17	V	0	0.83	0.57	0.68	0.79	0.85	1.01
Nov. 18–Dec. 9	VI	tr.	1.70	0.90	1.10	1.36	1.48	1.39
Average		tr.	1.53	1.23	1.78	2.28	2.21	2.89
Ammonia Nitrogen (in p.p.m.)								
Aug. 16–31	I	22.5	27.0	12.4	14.0	22.2	19.1	15.4
Sept. 1–20	II	31.8	10.5	1.5	5.3	8.0	17.6	5.1
Sept. 21–Oct. 10	III	34.6	16.5	3.3	9.4	15.3	22.0	11.4
Oct. 11–29	IV	67.3	27.6	7.5	13.1	37.4	24.6	23.8
Oct. 30–Nov. 17	V	51.0	22.9	2.5	5.0	19.0	31.1	15.5
Nov. 18–Dec. 9	VI	56.0	20.7	8.5	12.8	16.2	24.3	21.2
Average		43.9	20.9	6.0	9.9	19.7	23.1	15.4
Organic Nitrogen (in p.p.m.)								
Aug. 16–31	I	19.6	8.5	2.8	6.2	8.8	18.5	10.0
Sept. 1–20	II	20.2	4.9	3.2	2.4	2.7	10.4	6.5
Sept. 21–Oct. 10	III	9.2	5.5	4.2	5.9	5.9	7.5	6.6
Oct. 11–29	IV	8.2	18.7	11.2	10.6	15.8	4.7	4.0
Oct 30–Nov. 17	V	9.8	9.2	5.4	5.7	7.9	10.3	7.5
Nov. 18–Dec. 9	VI	9.3	11.7	5.6	8.3	7.4	8.7	16.0
Average		12.4	9.7	5.4	6.5	8.1	10.0	8.4

65° F. The effluents were stable, the 1 in. and ¾ in. rings showing over 98 per cent relative stability and the other filling materials producing effluents of 95 to 96 per cent average relative stability.

As may be seen from Table III the ¾ and 1 in. Raschig rings produced effluents with the lowest B.O.D. and the highest purification on

the basis of B.O.D. removal. Thus the ¾ and the 1 in. Raschig ring effluents showed reductions of 93.7 and 91.2 per cent B.O.D. by filtration and settling, and 82.6 and 77.2 per cent by filtration alone. Standard rock under the same conditions showed reductions of 87.3 per cent by filtration and settling and 71.3 per cent by filtration alone. Purification efficiencies with the 2½ in. Raschig rings and the Straight's block were about equal to those obtained with the standard crushed rock.

A consideration of the analyses of composite samples, given in Table III*A*, also indicates that the best results were obtained with the Raschig rings of the smaller sizes. Ammonia in the ¾ and 1 in. Raschig rings effluents was less than one-third to one-half that of the granite effluent. Organic nitrogen was also lower with the small rings. Nitrates were

TABLE IV.—*Effect of Filling Material on Reduction of B.O.D. (4 M.G.P.A.P.D.; 3 Min. Dosing Cycle)*

Date 1934–1935	Influent (Raw Sewage)	Filter Effluents											
		Granite		Raschig Rings								Straight's Block	
				¾″		1″		1½″		2¼″			
		Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.
Dec. 12	185	66	50	86	36	122	53	92	53	120	35	129	43
14	172	120	31	78	30	41	34	105	19	76	21	134	67
17	225	112	63	33	31	56	34	56	23	72	19	82	24
21	107	149*	32	36	51	72	39	74	27	125	29	107	70*
29	140	66	24	59	39	80	65	60	31	77	37	72	30
31	115	81	7	56	13	67	49	50	14	73	18	81	28
Feb. 14	215	99	84	75	60	79	69	109	81	127	113	76	65
18	225	123	86	51	33	70	35	105	75	138*	93	138*	84
20	277	99	73	81	73	96	83	144*	122	146*	141*	151*	117
22	212	146*	69	60	50	83	68	136	71*	148*	72*	145	72*
25	157	115	71.5	35	28	59	44.5	95	71.5	130	72.5	83	63
27	225	149	74*	36	32	89	62	119	74*	159*	75*	114	76*
Mar 1	212	108	71*	81	27	93	21	149*	44	135	75*	73	45
4	135	83	58	95	30.5	96	36	64	27.5	96	57	80	61.5
8	167	53	31	76	18	84	13	59	11.5	108	65.5	101	38.5
11	120	134	60	42	19	70	12	58	27	123	46	100	28
13	202	162*	77*	45	27	62	24	162*	77*	162*	77*	154	61.5
18	218	160	64.5	75	33	83*	29	156	40	161	70	168	65
25	220	425	12	130	15	136	17	173*	10	159	3	160	2
27	160	122	47	127	38	129	35	147*	50	146*	43	113	20
Apr. 1	180	157*	32	36	37.5	87	55	137	57	148	22	145	16
No. of analyses	21	21	21	21	21	21	21	21	21	21	21	21	21
Average	184.4	129.9	53.2	66.3	34.4	83.5	41.8	107.1	47.9	125.2	56.4	114.6	51.2
Average reduction		29.5	71.1	64.0	81.4	54.7	77.3	41.9	74.0	32.0	69.4	37.8	72.2

* O_2 completely depleted. Value shown is that of 100 per cent depletion.

TABLE IV*A*.—*Analyses of Preserved Composite Samples for Nitrogenous Constituents (4 M.G.P.A.P.D.; 3 Min. Dosing Cycle Series)*

Date 1934–1935	Comp. No.	Influent (Settled Ames Sewage)	Effluents					
			Granite 1 to 3″	Raschig Rings				Straight's Block
				¾″	1″	1½″	2¼″	
Nitrate Nitrogen (in p.p.m.)								
Dec. 10–Jan. 4	VII	tr.	7.6	21.1	15.9	10.9	5.2	11.8
Jan. 5–20	VIII*	tr.	7.3	17.6	10.4	11.4	6.4	8.9
Feb. 14–21	IX	tr.	1.7	2.7	2.4	1.8	1.4	1.3
Feb. 22–Mar. 11	X	—	2.2	8.8	5.2	2.5	1.1	.8
Mar. 12–Apr. 1	XI	—	1.7	20.5	20.5	8.1	3.7	3.8
Average		tr.	4.1	14.1	10.9	6.9	3.6	5.3
Nitrite Nitrogen (in p.p.m.)								
Dec. 10–Jan. 4	VII	tr.	1.33	1.70	1.59	1.55	1.18	1.59
Jan. 5–20	VIII*	tr.	1.36	1.83	1.48	1.58	1.35	1.56
Feb. 14–21	IX	Ft. tr.	tr.	.34	.36	.18	tr.	tr.
Feb. 22–Mar. 11	X	Ft. tr.	.15	1.23	.68	.40	tr.	.17
Mar. 12–Apr. 1	XI	.02	.99	2.18	1.95	1.80	1.26	1.77
Average		tr.	.76	1.45	1.21	1.10	0.76	1.02
Ammonia Nitrogen (in p.p.m.)								
Dec. 10–Jan. 4	VII	63.2	59.4	26.0	29.1	40.9	39.2	29.2
Jan. 5–20	VIII*	51.0	42.4	23.2	29.0	39.2	40.0	33.2
Feb. 14–21	IX	58.0	51.3	51.3	45.4	50.7	52.7	50.7
Feb. 22–Mar. 11	X	51.8	43.0	37.2	44.8	48.0	42.2	43.2
Mar. 12–Apr. 1	XI	46.0	44.0	17.0	23.6	34.0	44.8	32.8
Average		54.0	48.0	30.9	34.4	42.6	43.8	37.8
Organic Nitrogen (in p.p.m.)								
Dec. 10–Jan. 4	VII	15.6	9.4	7.2	7.0	7.2	12.7	14.0
Jan. 5–20	VIII*	13.8	28.8	22.4	25.2	25.8	31.0	21.0
Feb. 14–21	IX	20.5	12.4	11.2	9.6	13.8	13.0	11.2
Feb. 22–Mar. 11	X	17.7	15.0	13.2	14.5	14.4	13.8	16.4
Mar. 12–Apr. 1	XI	17.0	28.8	15.5	17.7	22.3	20.4	29.7
Average		16.9	18.9	13.9	14.8	16.7	18.2	18.5

* Severe cold spell causing units to freeze up. Not thawed out until Feb. 14.

about three times as high in the small ring effluents as in the granite effluent. In general, the granite effluents were very similar to those obtained from the 2½ in. Raschig ring and the Straight's block.

In Tables IV and IV*A* are shown the data for B.O.D. on the fresh 24-hr. composites and the preserved composite samples for the period December 19, 1934, to April 2, 1935, when the filters were operated at 4 m.g.p.a.p.d.

With respect to the relative efficiency of the various filling materials the results are similar to those observed with the 2 m.g.p.a.p.d. rate. The ¾ and 1 in. Raschig rings were superior to any other filling materials and the granite effluent was very close to that obtained by the 2¼ in. Raschig rings and the Straight's block.

The average temperature of the applied waste during this period was 52.2° F.

The ¾ and 1 in. Raschig rings showed relative stabilities of 83 per cent and 52 per cent, the granite 42 per cent, the 2¼ in. Raschig rings 31 per cent and Straight's blocks 48 per cent.

Nitrification was very satisfactory with the ¾ and 1 in. rings and fair with the 1½ rings, the quantities of nitrates produced being 14.1 p.p.m., 10.9 p.p.m. and 6.9 p.p.m., respectively. The granite showed 4.1 p.p.m., Straight's blocks 5.3 p.p.m. and the 2¼ in. Raschig rings 3.6 p.p.m.

Free ammonia and organic nitrogen determinations were lowest with the small size Raschig rings and highest with the granite and 2¼ in. Raschig rings.

The analyses of the applied wastes and the filter effluents during the period April 2, 1935, to June 18, 1935, with the filters operating at 8 m.g.p.a.p.d. are shown in Tables V and V*A*.

The average temperature of the applied waste during this period was 56° F. The effluents from the granite and 2¼ in. Raschig rings were not stable whereas the other filling materials showed surprisingly high relative stabilities. The ¾ and 1 in. Raschig rings showed relative stabilities of over 95 per cent.

From Table V it will be seen that the reduction in B.O.D. of the settled samples was only slightly better in the ¾ and 1 in. rings than in the granite. In this particular set of observations the Straight's block and the ¾ in. rings gave the highest B.O.D. reduction. It will also be noted that the difference between the settled influent and the effluent as it leaves the filter was very much greater in this series than at the 2 and 4 m.g.p.a.p.d. rates.

On the basis of the analyses of the composite preserved samples it will be noted that nitrification was very distinctly better on the ¾ and 1 in. Raschig rings, and even with the 1½ in. Raschig rings, than was observed with any of the other filter media. The variations in free ammonia and organic nitrogen content of the various effluents were not as great as were observed with the lower rates of filtration.

The analyses for nitrogenous constituents of the preserved composite samples during the operation period July 3 to Nov. 2, 1935, when filters were being charged at 16 m.g.p.a.p.d., are shown in Table VI, and the B.O.D. determinations are given in Table VI*A*.

The relative efficiency of the various filling materials at the 16 m.g.p.a.p.d. rate were similar to those observed with 8 m.g.p.a.p.d. The ¾ and 1 in. Raschig rings showed better reductions of B.O.D. and particularly better nitrification. It is also desired to call to attention

that the relation between the B.O.D. of the effluent as it leaves the filter, and the settled effluent, was similar to that observed for the 8 m.g.p.a.p.d. rate. This will be discussed in more detail later.

TABLE V.—*Effect of Filling Material on Reduction of B.O.D.*
(8 M.G.P.A.P.D.; 3 Min. Dosing Cycle)

Date 1935	Influent (Raw Sewage)	Filter Effluents											
		Granite		Raschig Rings								Straight's Block	
				¾″		1″		1½″		2¼″			
		Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.
Apr. 3	205	136	65	134	58	141	64	140	64	140	61	140	47
8	145	134	56	150	67	150	72	149	68	135	49	148	66
10	255	150	78*	158	78*	158	78*	158	78*	148	78*	158	67
12	150	151	90	146	73	155	73	138	73	128	32	151	54
17	185	102	80	132	63	105	45	76	48	86	29	98	16
19	190	96	62	115	35	98	31	80	40	90	19	92	15
22	260	51	22	85	21	71	9	50	9	62	6	61	11
24	190	88	62	98	38	109	46	113	59	110	14	121	25
26	160	65	43	62	8	82	26	78	37	93	8	73	20
29	260	60	51	51	34	55	37	52	30	70	12	88	14
May 1	260	96	43	77	24	97	30	106	31	100	28	102	12
6	205	137	75*	136	75*	139	75*	138	75*	128	50	142	51
8	280	93	32	111	39	125	46	94	41	94	27	113	15
13	290	96	31	155	48	123	42	92	32	79	27	112	37
17	260	144*	72*	144*	72*	144*	72*	144*	72*	144*	72*	144*	62
20	100	77	33	94	32	77	25	68	38	77	36	107	41
22	210	100	56	133	65	128	41	84	55	124	45	128	73
29	165	76	35	77	12	110	14	77	5	81	30	100	25
June 3	85	113	24	77	13	103	21	77	23	101	23	74	17
5	175	78	20	45	6	84	9	69	13	68	15	68	10
7	120	104	20	42	15	58	17	57	22	89	24	74	29
10	105	124	22	70	17	88	13	91	25	119	19	95	24
12	160	86	28	64	18	105	16	71	27	59	13	82	21
17	120	58	24	42	23	37	17	37	20	56	20	87	27
No. of analyses	24	24	24	24	24	24	24	24	24	24	24	24	24
Average	189	101	46.8	100	38.9	106	38.3	93.7	41.0	99.8	30.7	106	32.5
Average per cent reduction		46.5	75.2	47.1	79 4	43.9	79.7	50 4	78.3	47.2	85.8	43.9	79.1

* O_2 completely depleted. Value shown is that of 100 per cent depletion.

The effluents were not stable. The temperature of the applied waste was 62.3° C.

TABLE VA.—*Analyses of Preserved Composite Samples for Nitrogenous Constituents (8 M.G.P.A.P.D.; 3 Min. Cycle Series)*

Date 1935	Comp. No.	Influent (Settled Ames Sewage)	Effluents					
			Granite 1 to 3″	Raschig Rings				Straight's Block
				¾″	1″	1½″	2¼″	
Nitrate Nitrogen (in p.p.m.)								
Apr. 2–23	XII	tr.	2.0	16.5	12.2	11.7	0.3	2.2
Apr. 24–May 16	XIII	tr.	1.5	8.5	8.8	4.5	1 3	3.9
May 17–June 16	XIV	—	3.1	11.9	9.0	5.2	2.1	1.8
June 7–June 18	XV	—	6.8	17.0	13.3	8.2	4.0	8.5
Average		—	3.3	14.0	10.8	7.4	1.9	4.1
Nitrite Nitrogen (in p.p.m.)								
Apr. 2–23	XII	0.14	.08	3.20	1.02	1.04	0.11	2.12
Apr. 24–May 16	XIII	tr.	.38	1.30	1.34	.99	1.30	1.02
May 17–June 16	XIV	0.15	1.66	3.48	2.34	1.44	1.44	1.35
June 7–June 18	XV	0.29	2.03	2.60	2.13	2.13	1.53	2.45
Average		0 15	1 04	2.64	1.71	1.40	1.09	1.73
Ammonia Nitrogen (in p.p.m.)								
Apr. 2–23	XII	54.0	52.0	33.0	40.4	49.0	64.0	38.0
Apr. 24–May 16	XIII	33.2	32.6	38.5	39.0	27.8	25.0	33.0
May 17–June 16	XIV	21.4	28.7	14.9	18.0	20.0	21 3	20.7
June 7–June 18	XV	21.6	22.2	8.3	11.6	17.6	19.4	15.8
Average		32.6	33.9	23.7	27.2	28.6	32.4	26.9
Organic Nitrogen (in p.p.m.)								
Apr. 2–23	XII	16.5	17.0	12.0	16.6	13.3	14.0	15.9
Apr. 24–May 16	XIII	19.6	13.3	21.4	14 6	18 2	14.1	13.6
May 17–June 16	XIV	8.0	17.0	8.8	12.0	8.8	10.7	11.6
June 7–June 18	XV	10.4	19.8	9.5	11.7	14.7	12.0	18.0
Average		13.6	16.8	12.9	13.7	13.8	12.7	14.8

DISCUSSION

In considering the results of the observations reported in this paper, it should be borne in mind that the individual runs with the different rates of application were for relatively short periods. Experiments at rates of 2, 4 and 16 m.g. per acre per day were observed for four months and the 8 m.g. series for two months. The data should therefore be considered as suggestive and as pointing the way to a more extensive and prolonged series of experiments.

The small size of the experimental units, 2 ft. square, and the fact that in one of these runs (the 4 m.g. series) the filters actually froze, must also be considered in evaluating the data.

TABLE VI.—*Effect of Filling Material on Reduction of B.O.D.*
(16 M.G.P.A.P.D.; 3 Min. Dosing Cycle)

Date 1935	Influent (Settled Ames Sewage)	B.O.D. of Effluents											
		Granite 1 to 3″		Raschig Rings								Straight's Block	
				¾″		1″		1½″		2¼″			
		Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.	Tot.	Set.
July 8	75	72	26	45	20	45	11	70	14	48	13	63	8
15	75	50	13	45	4	51	4	63	16	40	10	79	21
31	182	60	54	81	27	67	32	127	52	117	63‡	92	45
Aug. 1	125	29	21	37	15	29	17	56	28	73‡	55	49	12
17	176	128	67‡	94	23	129	33	98	48	142‡	45	142‡	67‡
23	310	154‡	73‡	84	41	128	50	124	73‡	154‡	73‡	154‡	73‡
26	265	156‡	74‡	74	23	61	22	113	55	131	74‡	149	74‡
28	350	159	82	116	69	90	31	153	75	205	105	230	127
30	263	170	132‡	213	103	127	85	215	130	190	68	230‡	133‡
Sept. 4	134	77	33	57	29	52	23	67	34	52	37	70	36
6	230	120	68‡	30	28	63	37	143	101	104	71	115	68‡
12	385‡	146	66	33	23	55	33	62	29	118	55	89	45
16	362	188	67	80	11	187	19	118	22	132	60	124	50
17	375	205	91	220	27	250	28	225	33	246	79	142	48
21	352	191	48	*	29	198	19	147	1	88	27	120	53
25	355‡	46	27	29	3	35	4	42	11	48	25	47	17
Oct. 4	260	191	40	141	18	84	8	181	28	139	33	124	55
5	195	91	28	78	17	56	11	90	17	62	27	72	18
7	189	75	58	53	10	57	19	52	23	61	36	46	24
9	246	80	26	118	20	118	22	69	24	84	34	158	24
10	210	65	38	68	14	97	17	32	8	55	28	60	18
11	160	68	21	78	15	57	16	33	11	63	9	64	19
14	198	233	42	221	22	224	18	63	10	165	24	190	12
16	185	233‡	66	232	20	233‡	25	233‡	37	225	46	233‡	31
18	195	154	22	168	14	192	18	203	23	137	25	127	20
22	298	218	36	225	29	149	34	252	30	217	41	231	43
23	154	63	26	93	19	88	19	81	18	53	28	24	20
25	195	112	60	110	17	99	24	131	22	85	29	90	31
30	143	203	27	233	15	235	20	215	21	179	20	132	31
Nov. 1	150	107	24	150	11	180	18	133	17	118	23	35	12
No. of analyses	30	30	30	29	30	30	30	30	30	30	30	30	30
Average	226	128	49	111	24	115	24	120	34	118	42	116	41
Average per cent reduction		43.4	78.6	51.1	89.4	49.4	89.6	47.1	85.1	48.0	81.4	48.8	81.8

* Broken.

‡ O_2 completely depleted. Value shown is that of 100 per cent depletion.

Average temperature of influent, 62.3° F.

TABLE VI A.—*Analyses of Preserved Composite Samples for Nitrogenous Constituents (16 M.G.P.A.P.D.; 3 Min. Cycle Series)*

Date 1935	Comp No.	Influent (Settled Ames Sewage)	Effluents					
			Granite 1 to 3″	Raschig Rings				Straight's Block
				¾″	1″	1½″	2¼″	
Nitrate Nitrogen (in p.p.m.)								
July 3–Aug. 1	XVI	0	1.02	1.92	1.08	1.00	1.10	1.68
Aug. 1–Aug. 27	XVII	*	tr.	0.78	tr.	tr.	tr.	tr.
Aug. 27–Oct. 3	XVIII	0	tr.	2.10	3.00	tr.	tr.	tr.
Oct. 3–Oct. 22	XIX	0	tr.	5.51	6.95	3.20	tr.	3.10
Oct. 22–Nov. 2	XX	0	tr.	8.00	6.40	tr.	tr.	1.90
Average		tr.	0.20	3.678	3.488	0.84	0.22	1.34
Nitrite Nitrogen (in p.p.m.)								
July 3–Aug. 1	XVI	0	0.280	1.500	1.18	0.310	0.520	0.490
Aug. 1–Aug. 27	XVII	*	tr.	1.150	tr.	0.280	tr.	0
Aug. 27–Oct. 3	XVIII	0	0.033	0.510	0.39	0.230	0.029	0.271
Oct. 3–Oct. 22	XIX	0	0.220	1.090	1.05	0.660	0.280	0.790
Oct. 22–Nov. 2	XX	0	tr.	0.067	0.047	0.045	0.027	0.067
Average		0	0.107	0.863	0.553	0.305	0.171	0.324
Ammonia Nitrogen (in p.p.m.)								
July 3–Aug. 1	XVI	12.8	15.8	10.8	12.0	12.36	12.0	11.0
Aug. 1–Aug. 27	XVII	*	21.0	14.4	15.2	15.15	16.6	15.6
Aug. 27–Oct. 3	XVIII	23.6	22.6	19.6	9.6	15.6	16.0	24.8
Oct. 3–Oct. 22	XIX	34.0	42.0	31.0	31.2	32.5	39.8	41.2
Oct. 22–Nov. 2	XX	27.8	32.6	23.8	25.0	27.4	26.2	28.4
Average		24.55	26.8	19.92	18.6	20.602	22.12	24.2
Organic Nitrogen (in p.p.m.)								
July 3–Aug. 1	XVI	13.6	8.72	8.56	11.12	10.24	9.6	8.8
Aug. 1–Aug. 27	XVII	*	9.8	10.0	8.8	7.2	8.4	8.5
Aug. 27–Oct. 3	XVIII	15.0	22.4	16.6	23.9	18.0	13.5	15.0
Oct. 3–Oct. 22	XIX	2.38	4.6	4.1	4.6	3.5	2.96	3.5
Oct. 22–Nov. 2	XX	16.0	11.6	9.5	10.3	13.8	13.1	14.4
Average		11.75	11.42	9.75	11.74	10.55	9.51	10.04

The temperature of the sewage is a very important factor. During the 2 m.g. rate (August 15 to December 10, 1934) the temperature of the sewage was 65° F. and during the 16 m.g. series (July 2 to November 1) it was 62.3° F. The temperature was very low during the 4 m.g. series, namely 52° F., and during the 8 m.g. series it was 56° F. The 2 m.g. and 16 m.g. series are perhaps more directly comparable with one another because of the closeness of the temperatures of the incoming sewage, the periods of operation were in each case, approximately four months, and both series were during the same season of the year.

TABLE VII.—*Effect of Rate of Application and Nature of Filling Material on Nitrification and Reduction of B.O.D. (Summary Table)*

Dosing Rate (m.g.p.a.p.d.)	Influent (Settled Ames Sewage)	Effluents					
		Granite 1 to 3″	Raschig Rings				Straight's Block
			¾″	1″	1½″	2¼″	
B.O.D. (Total) in P.p.m.							
2	186	56	35	46	60	61	59
4	184	130	66	84	107	125	115
8	189	101	100	106	94	100	106
16	226	128	111	115	120	118	116
B.O.D. (Settled) in P.p.m.							
2	—	24	12	17	25	23	21
4	—	53	35	42	48	56	51
8	—	47	39	38	41	31	33
16	—	49	24	24	34	42	41
Pounds B.O.D. Removed per Acre-Foot by Filtration							
2	—	361	420	389	350	347	353
4	—	300	656	556	429	329	384
8	—	980	992	925	1050	992	925
16	—	2180	2560	2480	2380	2400	2440
Pounds B.O.D. Removed per Acre-Foot by Filtration and Settling							
2	—	450	483	469	446	452	459
4	—	728	830	790	756	713	740
8	—	1580	1670	1680	1650	1760	1740
16	—	3950	4500	4500	4260	4080	4130
Per Cent Reduction B.O.D. by Filtration							
2	—	69.9	81.6	75.3	67.9	67.2	68.4
4	—	29.5	64.2	54.4	41.9	32.0	37.8
8	—	46.5	47.1	43.9	50.4	47.1	43.9
16	—	43.4	51.1	49.4	47.1	48.0	48.8
Per Cent Reduction B.O.D. by Filtration and Settling							
2	—	87.3	93.7	91.2	86.9	87.8	88.4
4	—	71.1	81.4	77.3	74.0	69.4	72.2
8	—	75.2	79.4	79.7	78.3	85.8	79.1
16	—	78.6	89.4	89.6	85.1	81.4	81.8
Nitrate-Nitrogen in P.p.m.							
2	0.4	6.2	18.6	16.9	8.6	3.4	7.4
4	Trace	4.1	14.1	10.9	6.9	3.6	5.3
8	Trace	3.3	14.0	10.8	0 4	1.9	4.1
16	Trace	0.2	3.7	3.5	0.8	0.2	1.3

The results with respect to the reduction of B.O.D. for the 4 m.g. series were rather erratic and this is attributed to the difficulties encountered with freezing which have previously been mentioned. The fact that B.O.D. determinations were not made on each day's composite

but only on two or three days a week, as time permitted, makes the determinations on the basis of B.O.D. not strictly comparable with those of the preserved composite samples, which contained aliquot portions of every day's run. Considering both the reductions in B.O.D. and the production of nitrates probably will serve to give a fair picture of the relative efficiencies of the various units.

The effects of rate of application and nature of filling materials on nitrification and reduction in B.O.D. are summarized in Table VII. For purposes of simplifying the discussion attention will be paid particularly to the granite, the 1 in. Raschig's rings, as representative of this filling material, and Straight's block. All of the materials effected very satisfactory reductions of B.O.D. None of the filters showed any evidence of clogging during the period of operation of over a year. It is therefore apparent that filters could be operated at considerably higher rates than has been generally believed to be feasible.

The per cent of reduction in B.O.D. decreased, of course, as the rate of application increased, but the total quantity of B.O.D. constituents removed materially increased with increasing rates of application. Thus, considering the granite filter, a B.O.D. reduction of 69.9 per cent at the 2 m.g. rate dropped to 43.4 per cent at the 16 m.g. rate of application. Similarly with the Straight's block a reduction of 68.4 per cent fell to 48.8 per cent and with the 1 in. Raschig rings a reduction of 75.3 per cent at the 2 m.g. rate dropped to 49.4 per cent at the 16 m.g. rate. On the granite, 1 in. Raschig rings and Straight's block there were removed 361 pounds, 389 pounds and 353 pounds of B.O.D. per acre-foot when the waste was applied at the 2 m.g. rate, whereas at the highest rate of application (16 m.g.) the corresponding reductions in B.O.D. were 2,180 pounds for the granite, 2,440 pounds for Straight's block and 2,480 pounds for the Raschig rings.

Considering the various sizes of Raschig rings it is evident that best results were obtained with the ¾ and 1 in. rings and that the larger sizes, 1½ and 2¼ in., produced results that were more nearly comparable to those obtained on the granite and Straight's block. This held true for every rate of application of sewage.

The foregoing figures referred to the reductions effected by the filter proper. If the efficiency of filtration followed by sedimentation of the effluent is considered, even more striking results were obtained. At the 2 m.g. rate the B.O.D.'s removed by crushed rock and the 1 in. Raschig rings and Straight's blocks were 450, 469 and 459 pounds per acre-foot of filter. At the 16 m.g. rate the corresponding reductions were 3,960 pounds, 4,500 pounds and 4,130 pounds per acre-foot for the granite, 1 in. Raschig rings and the Straight's blocks, respectively.

It is interesting to note that the high efficiency of the process at the high rates of application was due, to a considerable extent, to the colloider action of the filters. This colloider action of the filter is very well shown by considering the pounds of B.O.D. removed by filtration in comparison with the pounds removed by filtration followed by sedimentation. The results are summarized in Table VIII for the 1 in.

Raschig ring filter, which will serve to illustrate this action. The results with the other filling materials were similar.

TABLE VIII.—*Relation of Dosing Rate to Colloider Action of Trickling Filter (One Inch Raschig Rings)*

Rate of Application (g.p.a.p.d.)	Pounds of B.O.D. Removed (Calculated as per Acre-foot of Filter)		Ratio (Per Cent)*	Per Cent of Total B.O.D. Removed Due to Post-settling
	By Filter Only	By Post-settling		
2×10^6	389	80	20.5	17.0
4×10^6	556	234	42.0	29.6
8×10^6	925	755	81.8	44.8
16×10^6	2480	2020	81.5	44.9

* Per cent of B.O.D. removed by post-filtration settling to amount removed by filtration.

At the 2 m.g. rate there were removed 389 pounds of B.O.D. per acre-foot by the filter and 80 pounds of B.O.D. through sedimentation of the filter effluent. The amount of B.O.D. removed by sedimentation was 20.5 per cent of that removed by filtration and 17 per cent of the total B.O.D. removed.

At the 4 mg. rate there were removed 556 pounds of B.O.D. per acre-foot by filtration and an additional 234 pounds of B.O.D. by post-sedimentation, which is equal to 42 per cent of that removed by the filter.

At the 8 m.g. rate of application there were removed 925 pounds per acre-foot by the filter proper and an additional 760 pounds of B.O.D. by post-sedimentation which was equivalent to 81.8 per cent of that removed by the filter.

At the highest rate of application observed (16 m.g.) the filter removed 2,480 pounds per acre-foot, and post-sedimentation accounted for an additional 2,020 pounds or 81.5 per cent of that removed by filter proper.

To put it another way the B.O.D. removed by post-sedimentation accounted for 17 per cent of the total B.O.D. removed when the waste was applied at 2 m.g., 29.6 per cent when applied at 4 m.g. and 45 per cent when applied at 8 or 16 m.g. The high B.O.D. reductions at the higher rates of application were therefore due, in great part, to biophysical and biochemical transformation of the colloidal, and possibly part of the soluble sewage constituents, to readily settleable materials which were flushed out from the filter. Filters, therefore, to a considerable extent, act as colloiders at high rates of sewage treatment.

Figure 4 shows the pounds of B.O.D. removed by filtration and filtration followed by sedimentation against the rate of application of sewage. It will be noted that as the rates of filtration are increased there is a greater disparity between the two curves. This disparity between the curves increases as the colloider action of the filter increases.

Perhaps the most striking and regular differences between the various filling materials were observed in the process of nitrification. It

will be noticed in Table VII that for every rate of application, nitrification was at maximum with the ¾ and 1 in. Raschig rings, and decreased with the larger sized rings. In general, nitrification on the granite was slightly less than on the Straight's block and somewhat greater than on the 2¼ in. Raschig rings but less than that obtained with the 1½ in. Raschig rings.

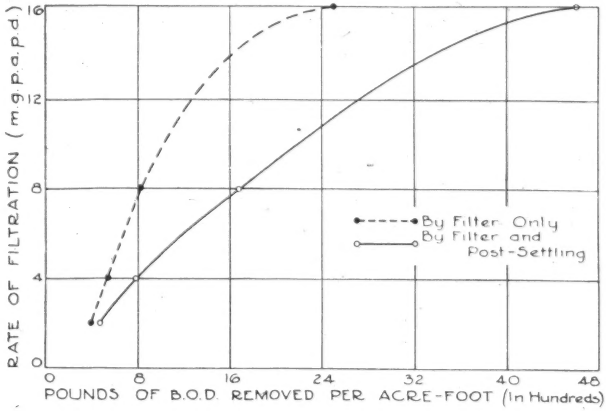


FIG. 4.—Relation of B.O.D. reduction by filtration and post-settling to rate of filtration.

As the rate of application increased, the production of nitrates decreased, the decrease being proportionately greater in the granite, the large size rings and the Straight's block than with the ¾ and 1 in. Raschig rings. Thus the granite effluent showed 6.2 p.p.m. of nitrates at the 2 m.g., 4.1 p.p.m. at 4 m.g., 3.3 p.p.m. at 8 m.g. and only 0.2 p.p.m. at the 16 m.g. rate. Similarly with the Straight's block 7.4 p.p.m., 5.3 p.p.m., 4.1 p.p.m. and 1.6 p.p.m. of nitrates were produced at the respective rates. The ¾ in. Raschig rings, on the other hand, showed 18.6 p.p.m., 14.1 p.p.m., 14.0 p.p.m. and 3.7 p.p.m. while the 1 in. rings showed 16.9 p.p.m., 10.9 p.p.m., 10.8 p.p.m. and 3.5 p.p.m. nitrates in their effluents when the filters were charged at rates of 2, 4, 8, and 16 m.g., respectively. Nitrification was therefore by far the best with the small size Raschig rings, from which it may be inferred that whereas the standard granite had probably approached its capacity, at least as far as nitrification was concerned, with respect to the rate of application of sewage, the small size Raschig rings could be charged at a still higher rate without completely eliminating nitrification. Nitrification seems to be related to the surface area of the filling materials. The relationship is shown graphically in Figure 5.

SUMMARY

Trickling filters employing various types of materials as filter media were charged with domestic sewage at rates of 2,000,000 to 16,000,000

gallons per acre per day. It was observed that considerable reduction in B.O.D. could be obtained at a very high rate (16 m.g.) on the filling materials considered, namely, granite, a specially designed perforated clay block and Raschig rings. At the 16 m.g. rate reductions of 43.4 per cent, 48.8 per cent, 49.4 per cent in B.O.D. were obtained on the

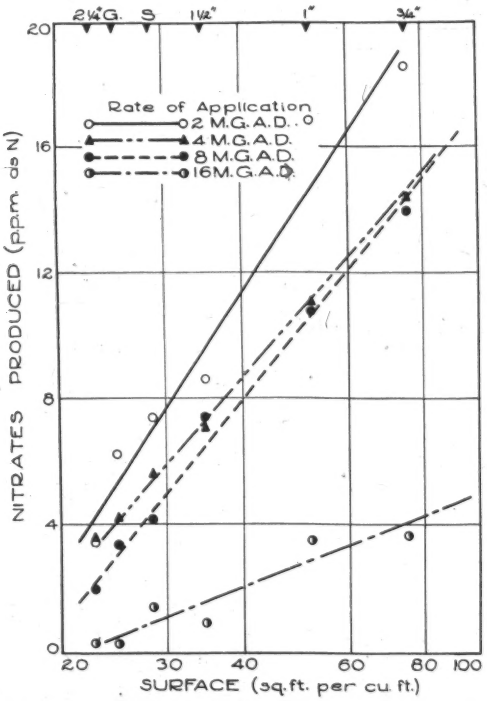


FIG. 5.—Relation of surface of filter medium to nitrification.

granite, the perforated blocks and the 1 in. Raschig rings respectively. After sedimentation of the filter effluents, the degrees of purification obtained were 78.6 per cent, 81.8 per cent and 89.4 per cent respectively, for the three filter media under consideration.

Nitrification was distinctly better with the small size Raschig rings (¾ to 1 in.) than with the other filling materials.

Considerably larger proportions of the filter effluents were in settleable form at the higher rate (16 m.g.) than were obtained with the customary rate now employed for sewage treatment (2 m.g.).

A considerable portion of the B.O.D. reduction by filtration and post-sedimentation is attributed to the colloider action of the filter.

This colloider action is particularly noticeable at the higher rates (8 to 16 m.g.). On the basis of pounds of B.O.D. removed per acre-foot of filter, the reduction in B.O.D. by post-sedimentation of the filter effluent was 20.5 per cent of the reduction obtained by the filter proper when the rate of application was 2 m.g.; the portion of B.O.D. constituents removed by post-sedimentation as compared with that removed by the filter rose to 42.0 per cent when the sewage was applied at 4 m.g. and to 81.8 per cent and 81.5 per cent with the 8 and 16 m.g. rates. The high B.O.D. reductions at the high rates are attributed, in great part, to the bio-physical and bio-chemical transformation of the colloidal and other sewage constituents into readily settleable material which was flushed out from the filter.

The B.O.D. removal per acre-foot of filter by filtration alone rose from 350 to 420 pounds at the 2 m.g. rate to 2,180 to 2,560 pounds at the 16 m.g. rate.

The B.O.D. removal per acre-foot by filtration and post-settling of the filter effluents increased from 450 pounds to 3,950 pounds with granite, and with Raschig rings it rose from 469 pounds to 4,500 pounds as the rate of application of sewage was increased from 2 to 16 m.g.

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THE DISPOSAL OF GARBAGE IN THE SEWERAGE SYSTEM *

By Lewis V. Carpenter, Albert C. Rogel, and Bernard Grabois

Associate Professor of Sanitary Engineering, New York University, Assistant in Civil Engineering, New York University, and Research Chemist, Works Progress Administration

Garbage is not a new factor in the sewerage system as all kitchen waste which will pass the sink strainer goes to the sewer and many people dispose of larger particles of food stuff through the toilet. An examination of the character of material collected on racks and fine screens will show much kitchen waste. We are at a loss to explain how some of the material gets into the sewers. Since one large company has introduced a small grinder which can be attached to the kitchen sink, and several other companies are marketing large grinders for use in central grinding stations, it has been proposed by some enthusiasts that the ultimate solution of the garbage disposal problem is through the use of the sewers. There is considerable merit to this proposal but certainly enough data are not available to warrant indiscriminate dumping of all garbage into the sewer without investigating the effect on the sewers, the sewage disposal plant and the streams.

Quantities of Garbage

Very few cities or municipalities keep accurate records of the actual amount of garbage collected. Herberger [1] in a survey of refuse collection in New York State found that the average garbage collected per capita per day was 0.678 pounds, with a maximum of 1.17 pounds and a minimum of 0.436 pounds. Figures show that within a given municipality the per capita variation of any year from the average for that municipality is not likely to be very great. He concludes that a municipality should keep accurate records for a year of the amount of garbage collected, rather than to estimate the production from average records of other cities. Rogel [2] weighed the garbage produced in a household of six persons in New York City and arrived at a value of 0.55 pounds of green garbage per day. Other authorities give values as high as 2.0 pounds per capita daily, but the average expected load should be about 0.5 pounds, with a seasonal deviation of about 50 per cent.

Home Treatment of Garbage

Home treatment of garbage is accomplished by the grinder attached to the sink. Where the municipality collects the refuse, it will be difficult to educate householders to separate the garbage from foreign material. In cities where garbage is disposed of by hog feeding or reduction, large amounts of metal and glass are found in the garbage. Unless such materials are removed before the garbage is ground, sewer

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velocities may have to be increased, probably grit chambers in existing plants may have to be extended, and sludge banks may form at sewer outlets, where the sewage is disposed of by dilution only. These problems are difficult to investigate because the characteristics of the sewerage systems and the methods of separating refuse are special problems peculiar to each municipality.

Three methods of introducing garbage into the sewerage system are recognized. In the first, the garbage is hauled to the sewage disposal plants to be ground and added directly to the Imhoff or digestion tanks. The second method requires haulage to grinders located throughout the city where the garbage, after grinding, is dumped into the sewers. In the third method the garbage is ground at the kitchen sink by means of an electric grinder built into the sink. This method offers the ideal solution because the garbage is finely divided while fresh and the amount of inert material is low. However, even if the price of the kitchen units become attractive, it is difficult to foresee installation of enough installations to eliminate the collection of garbage. Probably, there will be enough installed in the higher class buildings to cause sewage plant operators some concern. The problem must therefore be solved as to how much garbage can be adequately handled in the sewage plant.

Central grinding stations would be located on trunk sewers of adequate capacity and in sections of the city where the odors would not be offensive to the nearby residents. The cost of collection and haulage of garbage should be about the same with grinding as with incineration, as the grinding station would in most cases be located in the same section of town as the incinerator. The economics of the problem requires comparison of the cost of grinding, plus additional operating cost of the sewage disposal plant, with the cost of incineration. Calvert [3] estimated the cost of grinding and adding garbage to the sewage at Indianapolis as $0.40 per dry ton, but he presented no cost data for increased air consumption or solids disposal. The net cost of garbage disposal by reduction was $3.00 per dry ton. Griffin [4] estimated that the dry volatile matter in the garbage at Greenwich, Connecticut, was three times the weight of dry volatile matter removed from the sewage, and that addition of garbage to the primary treatment plant would not be successful. He stated that all of the experimental work to date has been in cities where the volatile matter in the garbage is not as great as that in the sewage. The estimated cost of incineration of garbage and rubbish was $1.71 per ton.

Keefer [5] states, "If it is assumed that garbage will digest and can then be dried as readily as sewage solids, the cost of treating garbage containing 25 per cent of solids would average $2.81 per wet ton. When the grinding charges are included, the total cost per ton under average conditions would be about $3.20 per ton. These figures do not include any further expense that might result from the production of additional screenings, grit, scum or grease. Moreover, the cost of possibly having to provide somewhat larger secondary treatment units has not been included."

Fox and Davis [6] on July 28, 1923, filed application for a patent on the process of grinding garbage and disposing of it with the sewage. The experiments were conducted on a small scale at Lebanon, Pa., where the garbage was ground in a bone mill and sent two miles through the sewer to the sewage Imhoff tanks. Technical information on the actual results accomplished at the installation have not been published.

Digestion of Garbage

A W. P. A. project under the direction of the Division of Sanitary Engineering, New York University, was initiated to study the effect of garbage on the digestion of sewage solids. Studies similar to this have been made elsewhere, and the results have shown that garbage may be successfully digested with sewage sludge, but the fact that the garbage used in these studies was ground by a kitchen waste unit seemed to justify further investigation.

It was desired to approximate the probable proportions of garbage and sewage of an average North American community, assuming that most of the garbage would be introduced into the sewerage system through the use of kitchen waste disposers. The proportions to be used were based partly on Prof. Fair's [7] suggestion that per capita values of garbage, fresh and digested sludges be considered in digestion studies. Table I gives per capita data taken from Prof. Fair's paper, with the last column supplied by the writers.

Table I.—*Per Capita Production of Garbage and Sewage Sludge*

Materials	Lb. per Year	Per Cent Moisture	Per Cent Volatile Solids	Lb. Dry Volatile Solids per Year
Green Garbage	180	75	90	40
Sewage Sludge	800	95	70	28
Digested Sludge	168	85	45	11

Fresh sewage sludge from sedimentation tanks and digested sludge were obtained from the Huntington, Long Island, sewage treatment plant. The analyses of the sludges and garbage used in the laboratory studies are given in Table II.

Table II.—*Analyses of Sludges and Garbage*

Material	Per Cent Total Solids	Per Cent Moisture	Per Cent Volatile Solids Dry Basis
Sewage Sludge	2.77	97.23	73.4
Digested Sludge	6.27	93.73	51.7
Green Garbage	25.00	75.00	90.0
Settled Garbage Sludge	2.80	97.20	90.6

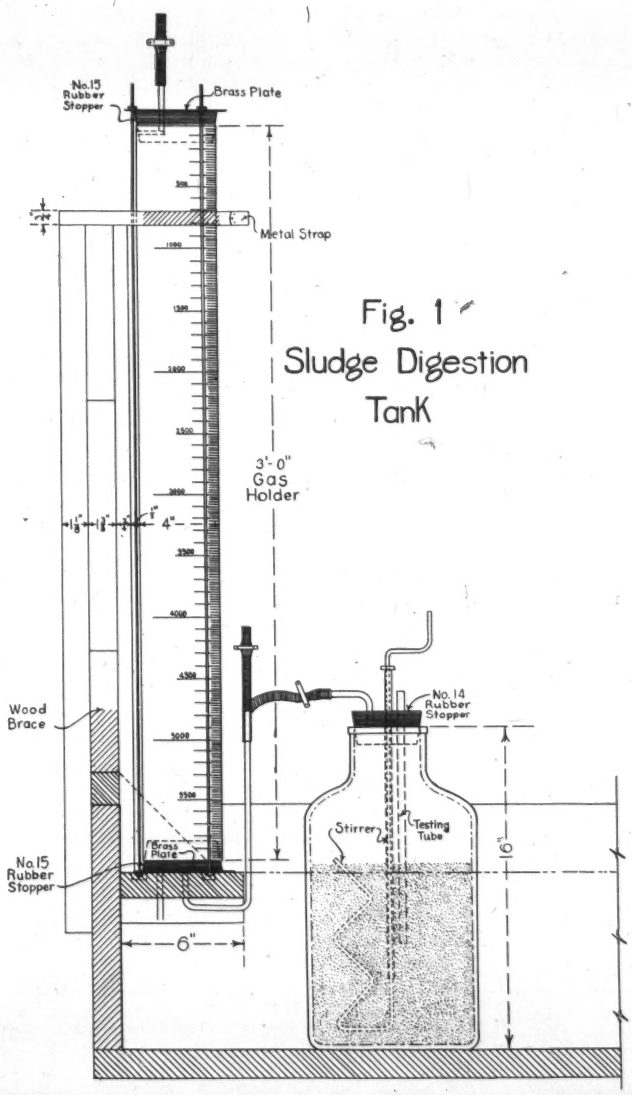
No.15
Rubber
Stopper
Brass Plate
Metal Strap
Fig. 1
Sludge Digestion
Tank
3'-0"
Gas
Holder
4"
No.14
Rubber
Stopper
Wood
Brace
Stirrer
Testing
Tube
16"
Brass
Plate
No.15
Rubber
Stopper
6"

FIG. 1

In order to approximate the quantity and characteristics of the garbage as it would reach the digestion tanks at the sewage works, it was essential that the garbage be settled after it had passed through the kitchen waste disposer. A charge of 1842 grams of green garbage, in about 30 gallons of water, was passed through the kitchen waste disposer and allowed to settle for one hour. This procedure gave 6480 grams of settled garbage sludge.

Figure 1 shows the apparatus used for digestion of the mixture and collection of the gas. An attempt was made to proportion the volatile solids in each digester to correspond with the volatile solids obtained from 100 gallons per capita per day of sewage and 0.5 pounds of green garbage. Garbage and sewage sludge were added to two digesters and sewage sludge only to two more. Table III gives the quantities of materials added to each digester.

TABLE III.—*Quantities of Materials in Digesters at Beginning. Digesters 1 and 2 (each)*

Materials	Grams Wet	Total Solids (gr.)	Volatile Solids (gr.)	Per Cent Volatile
Settled Garbage	2160	60.3	54.6	90.6
Sewage Sludge	4720	130.7	96.0	73 4
Digested Sludge	1165	72.9	37.8	51.7
Total	8045	263.9	188.4	71.4
Digesters 3 and 4 (each)				
Sewage Sludge	4720	130.7	96.0	73.4
Digested Sludge	1165	72.9	37.8	51.7
Total	5885	203.6	133.8	65.7

The digesters were placed in a constant temperature bath in which the temperature was maintained at 28° C. ($\pm$ 0.5° C.). Immediately after the digesters were charged and placed in operation, gasification commenced. Readings were taken daily of the gas production, temperature, barometric pressure and pH. It was found necessary to adjust the pH of the contents of all digesters. The pH values of the garbage-sludge mixture fluctuated more widely than the sludge alone and the former had a tendency to become more acid.

TABLE IV.—*Solids Content at Beginning and End of Digestion Tests*

Analysis	Mixture of Fresh and Digested Sludges			Mixture of Fresh and Digested Sludges Plus Garbage		
	Initial	Final	Per Cent Loss or Gain	Initial	Final	Per Cent Loss or Gain
Total Solids	203.6 gm.	161.2 gm.	20.8 Loss	263.9 gm.	195.5 gm.	25.9 Loss
Volatile Solids	133.3 gm.	36.9 gm.	35.0 Loss	188.4 gm.	110.4 gm.	41.4 Loss
Volatile, Per Cent	65.7	53.9	35.0 Loss	71.4	56.5	41.4 Loss
Ash	69.8 gm.	74.3 gm.	6.5 Gain	75.5 gm.	85.1 gm.	12.7 Gain

TABLE V.—*Gas Production Data (47 days digestion period)*

Materials in Digesters	Analyses (Per Cent by Volume)				Gross Heat Value B.T.U./ Cu. Ft.	Cu Ft. Gas per Lb. Volatile Matter Lost	Cu. Ft. per Lb. Fresh Vol. Matter	Lb. Gas per Lb. Vol. Matter Lost
	CO_2	CH_4	H_2	N_2				
Sewage Sludge + Digested Sludge ...	26.8	65.9	4.3	3.0	670	19.0	9.3	1.23
Sewage Sludge + Digested Sludge + Garbage.....	27.7	66.3	4.2	1.8	675	18.2	9.4	1.20

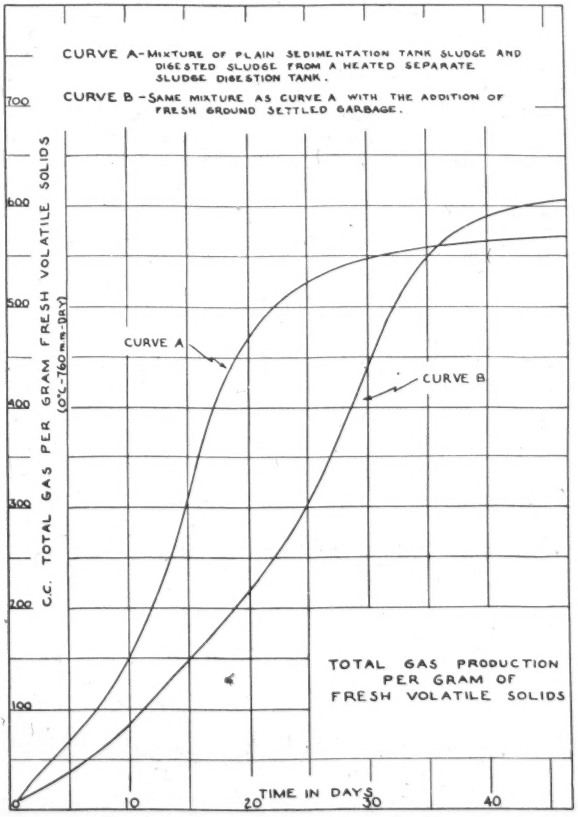

FIG. 2

At the start of the digestion, the seeded sewage sludge had a volatile solids content of 65.7 per cent, based on dry total solids, which was reduced to 53.9 per cent in 40 days, whereas the mixture of ground garbage, with the same quantity of seeded sludge, had a volatile solids content of 71.4 per cent at the start and 56.5 per cent after forty days.

The major part of the digestion, as indicated by the gas production curves (Fig. 2) of seeded sludge alone was completed in about 25 days, whereas that of the mixture of ground garbage and sewage sludge required about 40 days. At 25 days the sewage sludge produced 540 c.c.

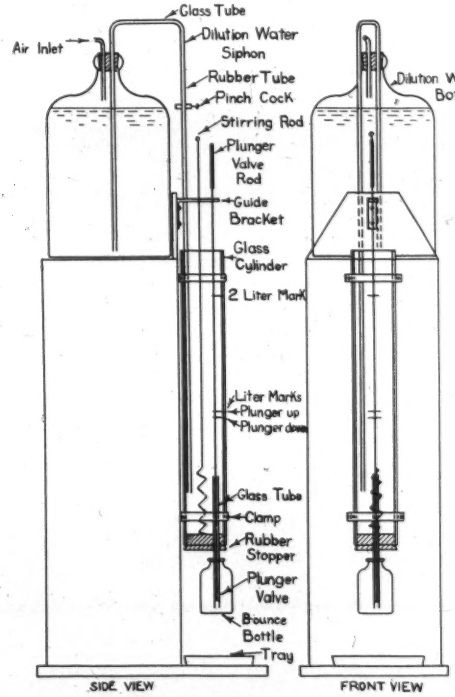

FIG. 3

of gas per gram of fresh volatile solids while the garbage-sewage sludge mixture produced 310 c.c. per gram. At 40 days, the values were 560 c.c. and 590 c.c. per gram, respectively. The batch method being employed, difficulty was experienced at the start in raising the pH of the garbage-sewage sludge mixture to the optimum, which probably accounts for the lag of the gas production. This difficulty would probably not be experienced in a plant where garbage and sewage solids were added daily.

There was no marked difference between the composition of the gases evolved by the seeded sewage sludge alone and those gases from the mixture of garbage and sewage. Table V shows that the two gases are essentially the same, with the same B.T.U. content, namely, 670 B.T.U. per cu. ft.

The digestion studies indicate that garbage can be satisfactorily digested with sewage sludge and that when compared on the basis of pounds of volatile matter, they are essentially the same.

B.O.D. of Garbage and Sewage

In determining the B.O.D. of ground garbage, using the ordinary siphoning method, the results were found to be worthless. A special piece of apparatus (Fig. 3) was constructed.

The apparatus consists of a glass cylinder dilution chamber which is clamped at a convenient height to a stand. A glass draw-off tube is held in place in the rubber stopper inserted in the bottom of the cylinder. Passing through this tube is a brass rod having at its lower end a valve made from a rubber washer and held between two nuts. This movable plunger-valve is prevented from being pulled through the top of the tube by the constriction at the top of the tube. It may be pushed out at the lower end, to allow the fluid to flow out. The upper nut is tapered to permit the rubber valve to be easily drawn back into the tube when the flow is to be stopped. The purpose of having the tube protruding up into the chamber is to assure a more representative sample. The valve is made to slide the length of the tube in order to return the liquid remaining in the tube to the chamber; thus, the next dilution is made with all of the liquid in the apparatus, and so precludes any faulty dilution. A wire stirring-rod is manipulated so that there is thorough stirring with no disturbance of the surface of the liquid which could cause aeration.

Table VI.—*Oxygen Demand of Garbage in Lb. Oxygen per Lb. Volatile Solids and Lb. Oxygen per Capita Daily*

Laboratory Analysis		Lb. O_2 Required per Lb. Volatile Solids	Lb. O_2 Required per Capita per Day
Volatile Solids (p.p.m.)	5-day B.O.D. (p.p.m.)		
7,320	4250	.582	.0655
10,200	6450	.632	.0712
9,000	6450	.717	.0808
12,000	6800	.567	.0638
7,340	4300	.587	.0660
	Avg.	.62 lb.	.07 lb.

Five different tests were made on the B.O.D. of ground garbage and the results are shown in Table VI. The values in Column 4 were derived using 0.5 lb. of green garbage as the average per capita daily

production. The 0.5 lb. of green garbage contained approximately 25 per cent total dry solids, of which 90 per cent was volatile, giving 0.125 lbs. total solids and 0.1125 lb. volatile solids on the dry basis. 0.1125 × lb. oxygen required per lb. volatile solids gives the oxygen demand in lb. per capita daily.

Table VII gives the pounds of oxygen required per capita per day as reported in U. S. Public Health Bulletin 173 and Bulletin No. 23, Illinois Water Survey.

TABLE VII.—*Oxygen requirements of Typical American Sewages (Based on 5-day B.O.D.)*

City	Pounds of Oxygen Required per Capita per Day
Reading, Pa.	0.099
Alliance, O.	0.100
Baltimore, Md.	0.106
Canton, O.	0.113
Fitchburg, Mass.	0.114
Rochester, N. Y.	0.119
Columbus, O.	0.148
Brooklyn, N. Y.	0.189
Chicago, Ill.	0.20
Schenectady, N. Y.	0.20

On the assumption that the per capita garbage production is 0.5 lb. the average oxygen requirement based on a 5-day B.O.D. was .07 lb. per capita daily, as compared with a value of approximately 0.2 lb. for sewage. If these data are comparable, garbage should increase the B.O.D. of the combined wastes by approximately 30 per cent, based on the per capita deductions.

A series of experiments was performed to determine whether the oxygen demand of a mixture of garbage and sewage solids was equal to the sum of the oxygen demand of the ground garbage and sewage sludge determined separately. Although not enough tests were made for definite conclusions, results seem to indicate that there is little or no difference between the oxygen demand of a mixture and the sum of the oxygen demands determined separately. Figure 4 gives the results of the laboratory experiments reduced to p.p.m. of oxygen demand per 1 p.p.m. of volatile solids. This graph was made as a composite curve of all of the results obtained in this series of experiments.

The 5-day B.O.D.'s per each p.p.m. of volatile solids of the sewage, garbage, and mixture of sewage and garbage, respectively, were 0.65, 0.76 and .68 p.p.m. The average of the sewage and garbage was 0.705 which compared with 0.68 for the mixture. These data are based on the raw sewage and garbage without settling and would represent in particular the increase in B.O.D. that could be expected if garbage were added to sewage and the mixture discharged into a stream or tidal estuary without treatment.

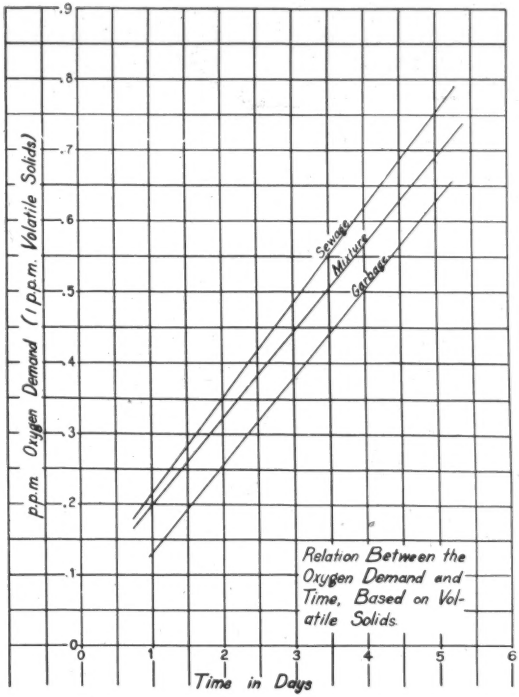

FIG. 4

SEDIMENTATION OF GARBAGE AND SEWAGE

Samples of mixed fresh ground garbage and sewage from the Dyckman Street screening plant were stirred for fifteen minutes in a mechanical stirrer so as to cause a minimum of aeration. The mixture contained about 60 per cent sewage volatile solids and 40 per cent garbage volatile solids. Liter samples were placed in Imhoff cones and in 2-inch diameter tubes, 36 inches long with a rubber stopper in the bottom, so that the sludge could be removed from the bottom. The sewage and sewage-garbage mixture was allowed to settle for two hours, the settling solids were recorded and the B.O.D. determined on the supernatant liquor. Parallel tests were made using the same sewage without the addition of ground garbage.

Table VIII gives the data on the total solids in the sewage and sewage-garbage mixture. It will be noted that in the sewage solids, the sum of the settling solids plus the solids in the supernatant equals

TABLE VIII.—*Total Solids in the Sewage and Sewage-Garbage Mixture*

Sewage (Total Solids)				Sewage Plus Ground Garbage (Total Solids)			
Crude p.p.m.	Settled p.p.m.	Imhoff c.c.	Supernat. p.p.m.	Crude p.p.m.	Settled p.p.m.	Imhoff c.c.	Supernat. p.p.m.
496	100	6.0	383	732	204	8.0	396
459	102	4.5	351	524	124	7.0	380
503	88	4.5	392	1403	612	29.0	771
425	123	8.0	301	801	328	14.0	432
298	71	5.0	227	464	160	12.0	294
655	140	5.5	506	1496	407	23.0	1011
Av. 472	104	5.6	360	903	306	15.5	547
	22.0%		76.1%		33.9%		60.6%

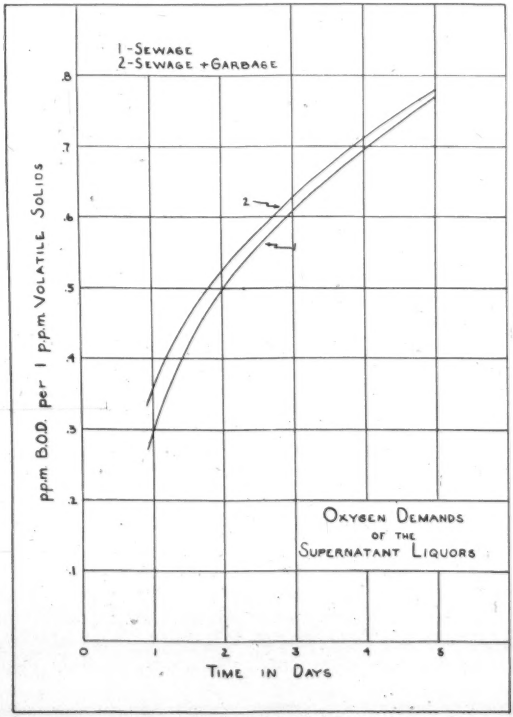

FIG. 5

98.1 per cent, while with the mixture the total is 94.2 per cent. These values should theoretically be 100 per cent but the floating material on the surface, which is removed before siphoning, plus any material which clings to the sides, would account for this discrepancy. An average of 33.7 per cent of settling solids in the sewage mixture, compared with 22 per cent with the sewage alone, confirms the well-established fact that the concentration of solids is a very important factor in sedimentation tank efficiency. Efficiency of solids removal is a very poor way to express the performance of a settling basin. The data show a greater settling efficiency in the case of the sewage-garbage mixture, but it should be noted that the total solids in the supernatant is higher in every case than when garbage was not added. The settlability tests indicate that the specific gravity of sewage sludge is the same as that of the mixed sludge. The addition of the garbage would result in a greater volume of sludge but the volume would essentially be the same as a sewage sludge of the same concentration.

The B.O.D. of the supernatant liquor showed very definitely that the B.O.D. of the settling tank effluent would be in direct proportion to the volatile solids, whether these were sewage or garbage volatile solids. Figure 5 shows the data on B.O.D. reduced to parts per million for each part per million of volatile solids. The data are summarized in Table IX.

TABLE IX

Sewage					Sewage and Garbage				
Crude	Supernatant Liquor				Crude	Supernatant Liquor			
Total Sol. p.p.m.	Total Solids p.p.m.	Vol. Solids p.p.m.	5-day B.O.D. p.p.m.	5-day B.O.D. per 1 p.p.m. Vol. Sol.	Total Sol. p.p.m.	Total Solids p.p.m.	Vol. Solids p.p.m.	5-day B.O.D. p.p.m.	5-day B.O.D. per 1 p.p.m. Vol. Sol.
496	383	203	166	.82	732	396	299	251	.84
459	351	277	155	.56	524	380	297	190	.64
503	392	254	200	.78	1403	771	631	504	.80
425	301	200	183	.92	801	432	328	278	.85
298	227	148	118	.80	464	294	221	181	.82
655	506	325	240	.74	1496	1011	809	599	.74
473	360	235	—	.77	903	546	431	—	.78

SUMMARY

The results of the tests presented in this paper indicate that it is feasible to handle ground garbage either in an Imhoff tank or in a separate sludge digestion unit. In calculating costs, it should be noted that the digestion tank capacity will have to be increased in size in pro-

portion to the increased volatile solids added with the garbage. Plants which have secondary treatment at the present time might utilize any excess capacity, but the plant that does not have excess capacity will find the effluent unsuitable after the addition of garbage. The B.O.D. increases in direct ratio with the increase in volatile solids. An equitable method of comparing incineration and digestion in the sewage disposal plant should charge the digestion of garbage with the proportionate cost of the entire sewage treatment process, based on the respective ratios of volatile solids. It is true that some plants have been designed for an expected growth in population and at the present time could utilize facilities that are not now being used, by the addition of garbage to the sewage. The digestion of garbage with sewage is still in the experimental stages and much work will have to be done on a plant scale before definite conclusions can be drawn.

The laboratory work can be summarized as follows:

1. Garbage ground by the kitchen wastes disposers can be digested satisfactorily in separate sludge digestion tanks.
2. The gas production per gram of fresh volatile solids was essentially the same with the addition of ground garbage.
3. There was no marked difference between the composition of the gases evolved by the seeded sludge alone and that of the mixture of garbage and sewage.
4. The gross heat value per cubic foot of gas was approximately 670 B.T.U. for both.
5. The B.O.D. of garbage and sewage in p.p.m. of volatile solids is essentially the same and if the estimated garbage production (0.5 lb. per capita per day) is added to the sewage, the B.O.D. of untreated mixture will be about 30 per cent greater than that of the sewage alone.
6. If the results are calculated on the basis of p.p.m. of volatile solids, the B.O.D. of the mixture is the sum of the B.O.D. of the sewage plus that of the garbage.
7. Garbage mixed with the sewage will settle readily but the total solids in the supernatant will be greater than without the addition of the garbage.
8. On settled sewage, the increase in B.O.D. of the supernatant is proportional to the increased volatile solids added.

Acknowledgment

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A STUDY OF THE RATIONAL DESIGN OF SETTLING TANKS *

By Thomas R. Camp

Associate Professor of Sanitary Engineering, Massachusetts Institute of Technology, Cambridge, Mass.

It is the purpose of this paper to describe briefly some of the factors influencing clarification by sedimentation in an effort to stimulate interest and discussion leading to improvements in the methods of design of settling tanks. A complete and rational theory of clarification by means of which removal can be predicted with accuracy does not seem possible of development for reasons which will be brought out later. Considerable improvement, however, can be made upon our present state of knowledge and methods of design by painstaking efforts to evaluate the effect of the various factors which influence settling of suspended particles.

Settling Velocities of Individual Particles

The natural starting point in the treatment of sedimentation theory is the consideration of the velocities of settling of individual particles in still water. When a particle is permitted to move without interference or rotation through a fluid due to the gravitational pull, the settling or rising velocity with respect to the fluid will become constant as soon as the resistance is equal to the weight of the particle in the surrounding fluid.

The law for the "resistance" or "drag" was first proposed by Newton as follows:

$$D = cA\frac{\rho v^2}{2} \tag{1}$$

in which

D = the drag,
c = the drag coefficient,
A = the projected area of the body in the direction of motion,
and $\frac{\rho v^2}{2}$ = the dynamic pressure.

The drag for a small sphere in a viscous fluid was first worked out by Stokes as follows:

$$D = 3\pi\eta dv \tag{2}$$

If this value of the drag is equated to the weight of the sphere in the fluid, $(\pi/6)d^3(\rho_1 - \rho)g$, the equation known as Stokes' law may be obtained as follows:

$$v = \frac{1}{18}\frac{g}{\eta}(\rho_1 - \rho)d^2 \tag{3}$$

* Presented before the 1936 Meeting of the Society for the Promotion of Engineering Education, Madison, Wis., June 24, 1936.

where (in the C.G.S. system)

v = the settling velocity in cm. per sec.,
g = gravity in cm. per sec.^{2},
η = absolute viscosity coefficient of the fluid in dyne sec. per cm.^{2} or poises,
ρ_1 = the density of the particle, gms. per cu. cm.,
ρ = the density of the fluid, gms. per cu. cm.,

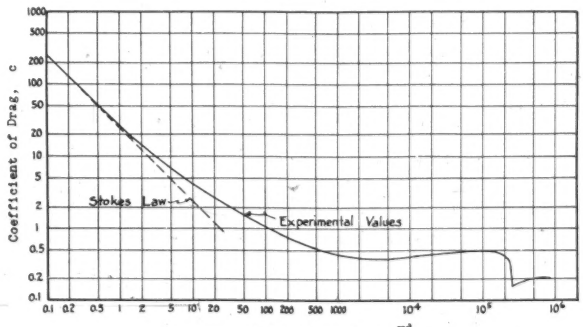
and

d = the particle diameter, cm.

The general equation for the settling velocity of a sphere in terms of the drag coefficient may be obtained by equating the drag of equation (1) to the weight of the sphere, from which

$$v = \sqrt{\frac{4}{3}\frac{g}{c}\frac{\rho_1 - \rho}{\rho}d} \qquad (4)$$

The drag coefficient c has been found by experiment to vary widely with the Reynolds' Number.[1] The drag is composed of both viscous and inertia forces. When the viscous forces predominate, "viscous" or "streamline" settling occurs. When the viscous forces are very small as compared to the inertia forces, the drag is entirely determined by the inertia forces. Stokes' law has been checked closely by experiment for values of Reynolds' Number less than 0.2. For values of Reynolds' Number from 0.2 to 1.0 the experimental values for the drag coefficient are



Drag Coefficient of Spheres in Fluids as a Function of Reynolds' Number
(After Schiller)

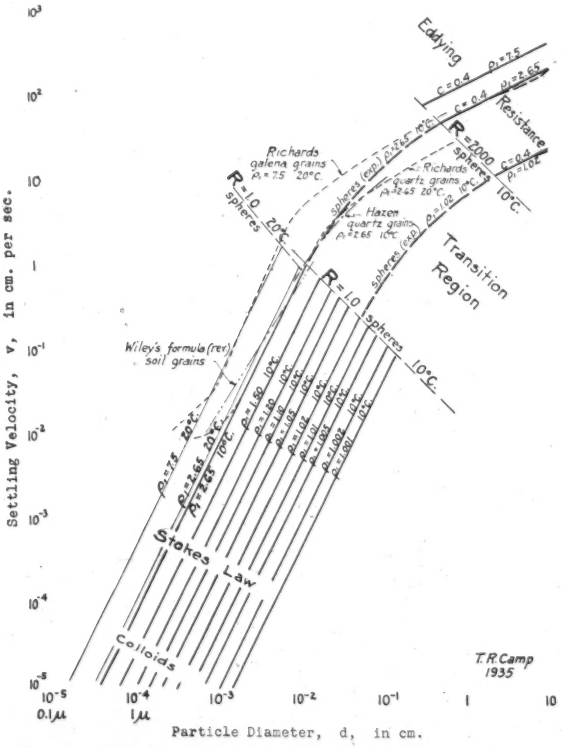
FIG. 1.

sufficiently near the values computed by means of Stokes' law to warrant the use of the law in this region.

The accompanying curve (Fig. 1) showing drag coefficient as a function

[1] Schiller, "Handbuch der Experimental Physik," Band 4, 2 Teil, 1932, pg. 369.

of Reynolds' Number is somewhat similar to the familiar curve of pipe friction factor as a function of Reynolds' Number. The region corresponding to the critical velocity in pipes is much more extended and gradual in the case of the moving sphere and is a stable region. The drag coefficient becomes practically constant at a value of about 0.4 for values of R from 1000 to 20,000. This region is known as the region of "eddying resistance" to distinguish it from the Stokes' law region. The intermediate region is a "transition" region in which both viscous and inertia forces are in play. Values beyond the region of eddying resistance are of little interest in sedimentation.



SETTLING VELOCITIES OF PARTICLES IN STILL WATER

FIG. 2.

The particles encountered in settling tanks are not spherical and usually are quite irregular. They vary in size, shape and density over a wide range. When settling in still water without interference and

without change in shape or in orientation they will settle at constant velocity. A change in orientation as by revolving will change the value of A in the drag equation and hence change the velocity. Flat particles such as mica tend to orient themselves in horizontal planes and hence to settle more slowly than bulky particles of equal mass and density. The settling velocity of bulky particles such as sand grains is practically equal to that computed by Stokes' law. Figure 2 shows some experimental settling velocities for quartz, galena and soil particles compared with velocities computed by means of Stokes' law.

Most of the particles of interest in sedimentation settle in the Stokes' law or transition region. Grit in sewage plants settles almost wholly in the transition region. Larger sewage particles and heavy activated sludge particles may also settle in the transition region, but for the most part the settling of individual particles in sedimentation basins in both water and sewage plants is within the Stokes' law region.

Clarification Theory for Ideal Continuous Flow Rectangular Basin

If a number of simplifying assumptions are made with regard to the type of flow in a basin and the character of the suspension, a rational theory of clarification may be developed, as will be shown below. The results obtained by this theory will not be in accord with experimental findings, but they will serve as a basis or yardstick by which basin performance may be measured. This theory was first published in a discussion (*Proc.*, Am. Soc. Civ. Eng., February, 1936) of the paper "Sedimentation in Quiescent and Turbulent Basins" by Professor Slade.

The simplifying assumptions for the "ideal" basin are as follows:

1. The direction of flow is horizontal and both direction and velocity are the same in all parts of the basin.
2. The concentration of suspended particles of each size is the same at all points in the vertical plane perpendicular to the direction of flow at the basin inlet.
3. All suspended particles maintain their shape, size and individuality during settling and settle without interference. Hence each particle is assumed to settle at constant velocity.
4. A particle is removed when it strikes the bottom.

In view of these assumptions each particle will settle in a straight line, and the direction and velocity of the path will be the vector sum of the liquid velocity and the particle settling velocity. All particles having the same settling velocity will move along parallel paths [See Fig. 3(*a*)]. Particles which settle parallel to the path from the surface at the inlet end to the bottom in the length l will be distributed uniformly along the bottom for the distance l and will thus be just 100 per cent removed in this length of tank. The velocity of such particles may be designated as v_0, and since it is equal to the discharge divided by the surface area Q/A it is also the overflow rate of the tank of length l. Particles settling along steeper paths are 100 per cent removed in less than the length l of the basin. Particles settling at velocities less than v_0 are only partially removed in

this length, and the removal is equal to the ratio of the settling velocity to v_0 as follows:

$$r = \frac{v}{v_0} = \frac{Av}{Q} \qquad (5)$$

where

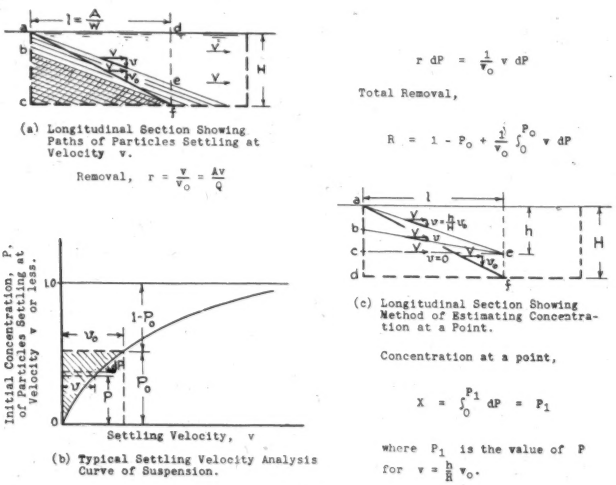
r = removal, as a ratio, of particles settling at any velocity v less than the overflow rate v_0,

A = the surface area of the tank corresponding to the length l,

and

Q = the rate of discharge.

From the above equation, it may be seen that for the ideal conditions assumed, the removal is independent of the depth of the basin for a given



(a) Longitudinal Section Showing Paths of Particles Settling at Velocity v.

(b) Typical Settling Velocity Analysis Curve of Suspension.

(c) Longitudinal Section Showing Method of Estimating Concentration at a Point.

FIG. 3.

discharge; and for particles which settle at velocities less than the overflow rate the removal is directly proportional to the surface area of the tank for a given discharge or inversely proportional to the tank overflow rate. Since the removal is independent of the depth for a given discharge it is also independent of the detention period. These relations were pointed out by Hazen [2] more than 30 years ago. They apply only to the case, however, where each suspended particle maintains its individuality and settles at a constant velocity. The conditions in an actual settling tank may differ greatly from the assumed conditions.

In order to estimate the removal for a suspension in which the settling velocities vary, it is necessary to have an analysis of the suspension in terms of settling velocities. Such a "velocity analysis" curve is analogous

[2] Allen Hazen, "On Sedimentation," *Trans.*, Am. Soc. C. E., 1904, pg. 45.

to a "sieve analysis" curve for sand. In plotting an analysis curve it is convenient to use settling velocity, v, as abscissa and initial concentration P of particles with velocities of v or less as ordinate, P being expressed as a ratio to total concentration [See Fig. 3(*b*)].

The value of P corresponding to the overflow rate v_0 may be obtained from the curve and is here designated as P_0. Since all particles which settle faster than v_0 will be 100 per cent removed, the removal of such particles in terms of the total suspension will be $1 - P_0$. The removal of particles having any settling velocity v less than v_0 is given by equation (5) and since the ratio of such particles to the total is dP, the removal in terms of the total suspension is

$$rdP = \frac{1}{v_0} vdP \tag{6}$$

The total removal of all particles is therefore

$$R = 1 - P_0 + \frac{1}{v_0} \int_0^{P_0} vdP \tag{7}$$

It is not necessary to assign an equation to the velocity analysis curve in order to solve for the removal. The last term of the above equation is the average vertical distance from the curve up to the horizontal line for $P = P_0$.

The concentration of suspended matter at any point in the hypothetical basin may be estimated from the position of the point and the "velocity analysis" curve of the original suspension [See Fig. 3(*c*)]. Since all particles settling at a given velocity v move along parallel paths, the concentration of such particles at any point which they can reach is the same as the initial concentration dP. Only particles which settle at velocities between zero and $(h/H)v_0$ can reach the point in question, h/H being the ratio of the depth of the point to the total depth of the basin and v_0 being the overflow rate of the partial basin between the inlet and the point. If the value of P corresponding to the velocity $(h/H)v_0$ be designated as P_1, the concentration of suspended matter at any point in terms of the initial concentration will be

$$X = \int_0^{P_1} dP = P_1 \tag{8}$$

which may be read directly from the velocity analysis curve for any value of $(h/H)v_0$.

Since values of P increase with v, it is evident that P_1 will increase with $(h/H)v_0$. Therefore in the hypothetical settling basin the concentration of suspended particles will increase with the depth below the surface. If lines of equal concentration of suspended matter be plotted on a longitudinal section through the basin, they will be straight lines meeting at the water surface at the inlet end. The shape of the velocity analysis curve will affect only the position of these straight lines.

Experimental Determination of Settling Velocity Analysis Curves

In order to apply the above theory to an actual settling tank, a settling velocity analysis curve of the suspension to be settled is required. Such an analysis may be made by permitting a portion of the suspension to settle quiescently in a container, such as a glass tube, from which samples for concentration tests may be drawn at measured depths at the end of various time intervals.

At this point in the procedure, the first obstacles to the application of the theory are encountered. Most of the suspensions dealt with by sanitary engineers flocculate during settling to such an extent that the assumption that each particle settles at a constant velocity is gravely in error. Moreover, it will be found difficult to prevent the formation of convection currents in the settling container. Residual currents or eddies caused by the initial filling of the container will last for 15 to 20 minutes and will cause no great interference unless the suspension is one which settles rapidly. By proper precautions in filling the container and controlling temperatures the effect of currents may be minimized. If, however, the suspension settles very slowly, such as is the case with fine clay, the test conditions are very unstable and the suspended particles may gather in masses or clouds which themselves tend to set up eddies.

If no flocculation or mixing takes place, the same settling velocity curve should be produced regardless of the depth in the container at which samples are drawn for concentration determinations. In dealing with iron floc, the author has found practically the same concentration in simultaneous samples drawn at different depths. Hazen, in the development of his theory of sedimentation, excluded flocculation but assumed no variation of concentration with depth. He assumed the cause of uniform concentration to be vertical mixing.

In order to check the validity of Hazen's assumptions, two similar suspensions of clay were allowed to settle in a constant temperature room at the Massachusetts Institute of Technology. One of these suspensions was peptized with sodium silicate in order to prevent flocculation. The peptized suspension gave approximately the same velocity curve for two depths, while the unpeptized suspension showed marked effects from slow flocculation and cleared much more rapidly.

Hazen's theory assumes that mixing retards settling. Undoubtedly it does when the mixing is violent. When the mixing is gentle, however, flocculation is promoted which will make clarification more rapid. In a number of cases in the author's studies in which two similar samples of iron floc were permitted to settle simultaneously, one settling quietly and the other stirred gently, the stirred sample invariably clarified more rapidly. Even in the absence of flocculation it remains to be proved that mixing, of the type existing in settling basins, will retard settling, that is, mixing *per se* and not short circuiting.

If in the test to determine the settling velocity analysis curve it is found that flocculation takes place to an extent sufficient to invalidate the theory, no settling velocity analysis can be made. In this case, however, concentration tests may be made upon samples drawn at the end of

various intervals of time from a number of different depths in the container. If the variation in concentration with depth is marked, the settling container used should preferably be as deep as the settling tank being studied. Predictions of removal can be made from results obtained in this manner, by consideration of the average concentration throughout the depth at the end of a period equal to the tank detention period. This method of approach with variations has been used recently in both England [3] and Russia,[4] and doubtless in this country. It is not new, but is subject to considerable improvement in technique.

Experimental Tank Studies

In order to approximate the assumptions made in the development of the theory, the author constructed a small rectangular tank 21⅝ in. wide with an average depth of 5⅝ inches. The inlet was equipped with a flume across the tank width discharging through orifices in the bottom and collection at the outlet was by means of a weir. In order to disperse the water uniformly over the tank cross-section area at the inlet and to collect it uniformly at the outlet, orifice walls were placed at both inlet and outlet ends extending over the full cross section area. The length of the tank between these walls was 27 inches. Discharge rates from about ¼ to 1 gal. per min. were used in the experiments [See Figs. 4 and 5].

A number of runs were made with iron floc in order to study removal, but the removals accomplished to date have been too small to permit any worth while analyses of the results.

Experiments were also made with brine solutions and potassium permanganate dye in order to study short circuiting. The results of these studies are of particular importance in this discussion. With both the brine, which could be seen through the glass side wall of the tank under proper lighting conditions, and the dye, the dispersion by means of the orifice wall at the inlet was excellent. The collection at the outlet end, being by a similar wall, was also excellent. The residual eddies caused by the inlet wall jets were dissipated within a distance of 2 to 6 inches from the wall in all cases.

Having procured uniform forward motion of the suspension at the inlet end and uniform collection over the cross section area at the outlet end, it might be expected that a uniform flow at constant velocity would obtain throughout the tank except for the influence of the drag on the tank walls and floor. In order to minimize the effect of drag the floor (a rubber belt) was made so that it could be moved at the tank velocity in the direction of flow. The expected results were not obtained. The flow was not uniform in any of the runs and in many of the runs velocities in the reverse direction and vertical velocities were observed which exceeded the nominal tank velocity. The salt solution, being heavier than water, invariably dropped to the bottom when beyond the influence of the inlet jets and proceeded in a thin layer along the bottom to the outlet end.

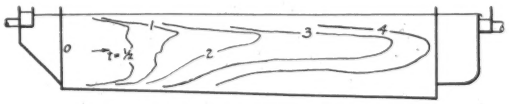
[3] Clifford and Windridge, "Experiments with Model Tanks," *Jrnl. and Proc. of the Institute of Sewage Purification*, Part I, 1935, pg. 136.

[4] Jukov, "Effect of Velocity of Flow in Settling Tanks on their Efficiency," Moscow, 1934.

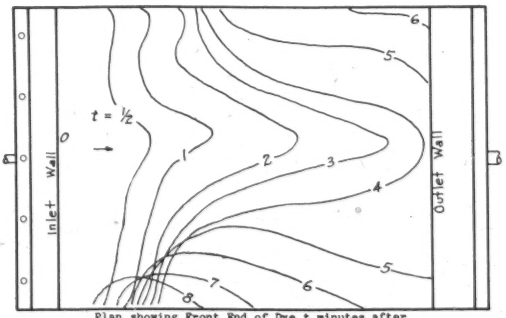
Since the path of flow was obviously changed by the salt, it was abandoned in favor of dye.

The dye was added as a solution at the tank inlet, and the time of appearance of the dye in the tank was noted as zero time. At the end of various intervals of time thereafter the position of the front of the dye as observed from above and also through the glass side wall of the tank was recorded [See Figures]. When the front of the dye passed through the effluent wall, samples of the effluent were collected in Nessler tubes at the end of various intervals of time until the dye had passed from the tank. The colors were compared as to concentration, and concentrations were plotted against time in order to study short circuiting [See Figs. 4 and 5].

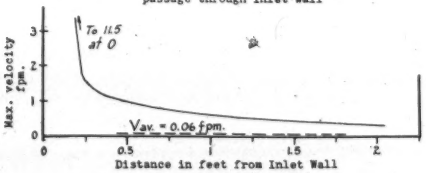
Short circuiting may be measured by the ratio of the time to the center of mass of the concentration curve over the detention period; by the ratio of the time for the first dye to appear in the effluent to the detention period; and by the time spread of the concentration curve. For ideal flow, both the above ratios should be unity and there should be no spread. The time of flow of the center of mass has been called by Capen [5] and by Hyde



Longitudinal Section showing Front End of Dye t minutes after passage through Inlet Wall

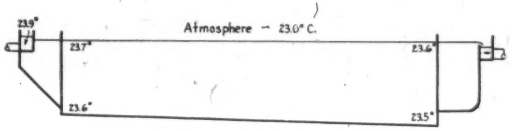


Plan showing Front End of Dye t minutes after passage through Inlet Wall

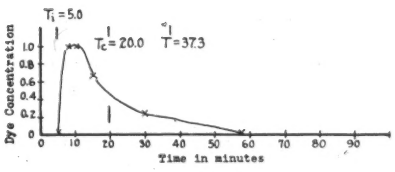


Distance in feet from Inlet Wall

[5] Capen, *Eng. News Record*, 99, 833 (1927).



Longitudinal Section showing Temperatures



Run No. 9
May 13, 1936
Both Orifice Walls in Place

$Q = 0.385$ gpm.

$V = 0.06$ fpm. $= 0.001$ fps.

$F = \frac{V^2}{Rg} = 1 \times 10^{-7}$

$\frac{T_i}{T} = 0.134$

$\frac{T_c}{T} = 0.535$

FIG. 4.

the "flowing through period," and Morrill [6] refers to the spread as "dispersion."

Short circuiting is usually thought of as being a function of the settling tank and the discharge. It is well known also, however, that if the incoming water is markedly different in temperature from that in the basin that it will short circuit to a greater extent either along the surface or along the bottom, depending upon whether it is warmer or cooler than the water in the tank. Short circuiting, therefore, is also a function of temperature differences.

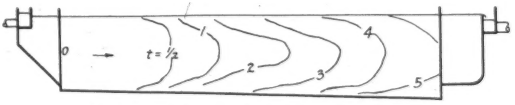
It might be assumed that if the temperature is practically constant, flow patterns and concentration-time curves can be duplicated if runs are repeated at the same discharge. This does not seem to be the case. In some of the attempts made by the author duplication was fairly close, but in others there seemed to be a complete lack of stability in the tank. Figure 5 shows the results of a run intended to duplicate the run of Figure 4. The results are quite different.

[6] Morrill, "Sedimentation Basin Research and Design," *J.A.W.W.A.*, Sept. 1932, pg. 1442.

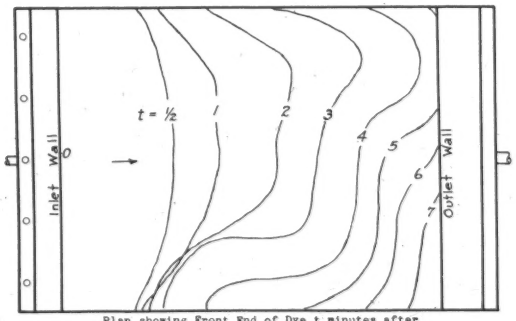
Factors Influencing Tank Stability

The front of the dye in the author's experiments never made the same pattern twice, even though temperature differences no greater than 0.1° C. existed. The character of flow, particularly at low nominal velocities, was a drift rather than a flow and was quite similar to the drifting of smoke from a cigarette in a quiet atmosphere. The motion may be in any direction, with the greatest tendency in the forward direction. Uniform forward motion over the entire tank cross section area was seldom approximated, even for a moment, beyond the zone of influence of the inlet wall. When such a desirable condition of flow was observed it was decidedly unstable. The most stable condition observed was when the front of the dye projected forward with a comet-like nose from the rest of the dye mass. With low velocities more than one of such noses may form.

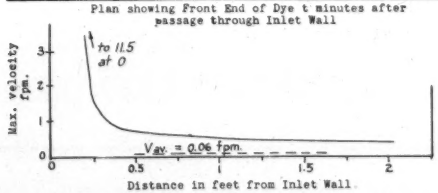
It has been the author's feeling that low tank velocities were desirable for the removal of particles with low settling velocities, in order that any vertical components of tank velocity which might exist would be too small to interfere seriously with the settling of the particles. It appears from



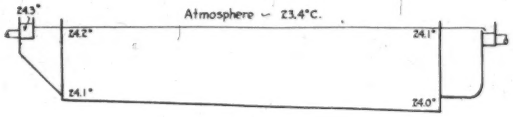
Longitudinal Section showing Front End of Dye t minutes after passage through Inlet Wall



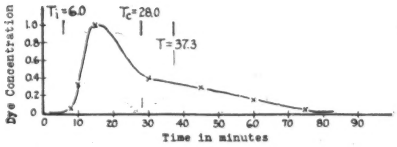
Plan showing Front End of Dye t minutes after passage through Inlet Wall



Distance in feet from Inlet Wall



Longitudinal Section showing Temperatures



Run No. 9A
May 13, 1936
Both Orifice Walls in Place

$$Q = 0.385 \text{ gpm.}$$
$$V = 0.06 \text{ fpm.} = 0.001 \text{ fps.}$$
$$E = \frac{V^2}{Rg} = 1 \times 10^{-7}$$
$$\frac{T_i}{T} = 0.16$$
$$\frac{T_c}{T} = 0.75$$

FIG. 5.

experimental observations, however, that there may be such a complete lack of stability at low velocities that vertical currents of relatively high velocity may appear without apparent cause. Evidently there are factors present which cause instability, and which have heretofore been given too little attention.

The variation in density of water due to differences in temperature is so small as to lead one to believe from a superficial examination that it may be neglected if temperature differences are small. For example, the density of pure water which is greatest at 4° C. is reduced by only 0.3 per cent by a rise in temperature to 25° C. A change of 1° C. at 25° C. changes the density by only 0.026 per cent and the changes are much less per degree at lower temperatures. In order to estimate the effects of temperature differences upon the relative motion of water, the author computed the velocities of small water spheres by means of Stoke's law, assuming that the temperature of the sphere and the surrounding water differed by only 0.1° C. The results as shown on Figure 6 indicate the possibility of very high vertical velocities due to relatively insignificant temperature differences.

If such small differences in density due to temperature differences can account for high vertical velocities, it follows that there are other causes which may result in vertical currents. Among these causes are differences in concentration of soluble substances and suspended particles. It has

been previously noted that clouds of suspended particles sometimes go down *en masse* during quiescent settling tests. Another cause is the liberation of dissolved gases. Computations similar to those above were made on water spheres containing small amounts of entrained air [See Fig. 6]. The results here are equally significant. A sphere as small as 1 mm. in diameter containing less than 5 p.p.m. of entrained air will acquire a velocity of more than 1 mm. per second.

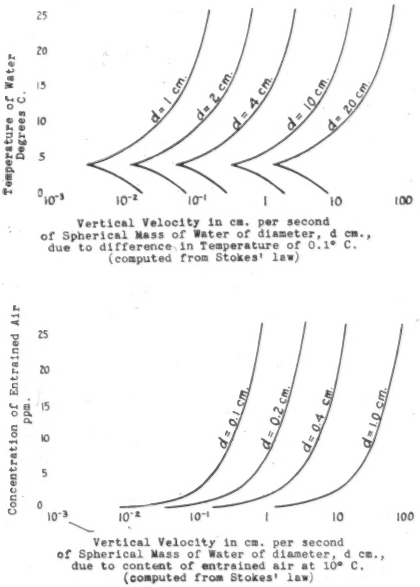


FIG. 6.

In this connection it is well to consider the effect of salt and dyes upon the flow patterns proposed to be measured by such salt and dyes. As previously stated, the author found that the salt solution changed the flow pattern so markedly that it had to be abandoned. The salt solution always moved along the tank bottom at high velocity until it reached the effluent wall, after which it was drawn off uniformly through the outlet wall along with the overlying water. The concentration curves with salt could be repeated with great accuracy, the flowing through ratio being 0.96 for three successive runs and the minimum time ratio being 0.14. In other words, the flow pattern formed by the salt was quite stable, but it was by no means identical with the flow pattern of the water without the salt.

The density of the dye solution was much closer to that of the water than was the density of the salt solution. In some cases where the dye

charge was heavy, changes in the flow pattern were observed. One such heavy charge resulted in a flowing-through ratio of 1.15, and a duplicate run following immediately with a lighter charge of dye produced a flowing-through ratio of 0.82. The two flow patterns were markedly different in vertical profile.

Relations of Stability to Hydraulic Characteristics

It would seem desirable in the design of settling tanks to aim at stability of flow; up to the point, that is, where further increases in stability result in decreased removal or efficiency. Towards this end the proper design of inlets and outlets to effect uniform entrance and exit conditions has been recognized. We may add also the recommendation that tanks be covered in order to minimize temperature changes and eliminate wind action. The effect of moving equipment for sludge removal, when the motion is not too fast, is still in doubt. Further questions which arise are with regard to the effect of tank velocities and dimensions.

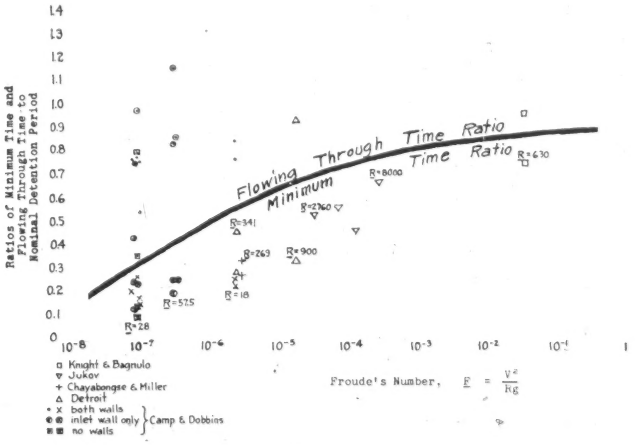


Fig. 7.

As pointed out previously, the dye fronts tend to take shapes analogous to the nose of a comet. If this shape is a stable one, it follows that high velocities in the center of the moving liquid and drag at the boundaries are characteristics of stability. Perhaps, then, the higher the velocity with relation to the distance between boundaries the more stable will the flow be.

In order to examine this hypothesis, the experimental work done by several investigators was studied with regard to the relation of the velocity-hydraulic radius ratio to stability as measured by both flowing-through ratio and minimum-time ratio. These studies tend to indicate the correctness of the hypothesis.

The forces acting in hydraulic problems are usually classified as inertia or those having to do with the motion of the liquid; viscous, or those having to do with drag or friction; and gravity, which are those having to do with weight. In our case, the unstabilizing forces appear to be gravity forces, and in order to effect stability it seems necessary to make the drag due to the forward motion of the liquid large in comparison with the gravity forces. This can be done by increasing the ratio of the inertia to the gravity forces. The ratio V/R tentatively used for a measure of the stability may be replaced by Froude's Number, V^2/Rg. A study of stability in terms of Froude's Number indicates a definite increase in stability as Froude's Number is increased [See Fig. 7]. The flowing-through ratio tends to fluctuate within a smaller range and to approach unity as the Froude Number is increased, and the minimum time ratio also rises markedly.

The experimental data shown in Figure 7 from Knight and Bagnulo [7] relate to measurements by means of salt on a small round-the-end baffled mixing chamber. The channels were about 1 in. wide and 3.6 in. deep, and the velocity was 0.2 ft. per second. Such excellent results could, of course, not be expected in a settling tank, nor would the Froude's Number for this experiment be approached in a conventional settling tank.

The experiments of Jukov [4] were made upon a settling tank 15 meters long by 1 meter wide and with an average depth of 0.86 meter. The measurements were made by means of dyes, the first appearance only of the dye having been observed. The nominal tank velocities were respectively 5, 10, 15, 20 and 29 mm. per second. Jukov was experimenting with the settling of paper mill wastes.

The experiments of Chayabongse and Miller [8] were made at M.I.T. using a steel channel equipped with one glass side. The tank was 23 in. wide by 18 in. deep and the velocity for the runs shown on Figure 7 was 0.45 ft. per minute. Dye was used.

The experiments at Detroit for the lower value of F were made by Fenkell and Wark.[9] The two points shown on Figure 7 give the limits of a large number of values of the minimum time ratio as determined by $KMnO_4$ dye. The experimental basins were 5.4 ft. wide by about 9 in. deep, and the velocity was about 0.4 ft. per minute. The points on Figure 7 for the higher value of F are for a single run with NH_4Cl solution using the same basins at a mean velocity of about 1.12 ft. per minute. This run was reported by Morrill.[6]

All of the experiments of Camp and Dobbins as shown upon Figure 7 were made upon the tank previously described, using $KMnO_4$ dye. The higher values of F were obtained by dividing the tank into 11 channels of equal width by means of vertical sheet metal baffles placed longitudinally.

The value of Reynolds' Number for each of the runs plotted on Figure 7

[7] Bagnulo and Knight, "A Study of Hydraulic Short Circuiting through Mixing Chambers," Bachelor's Thesis, M.I.T., 1936.

[8] Chayabongse and Miller, "An Experimental Study of the Velocity Distribution in a Model Settling Tank," Master's Thesis, M.I.T., 1933.

[9] Fenkell and Wark, Report No. F-18, Department of Water Supply, City of Detroit, November, 1928.

has been indicated on the figure. There appears to be no correlation between Reynolds' Number and stability. Experiments by Straub [10] tend to strengthen the conclusion that stability is not directly related to the value of Reynolds' Number. If, however, the velocity is increased in a tank of fixed dimensions in order to increase Froude's Number in the interest of stability, Reynolds' Number will also be increased. Since one of the most practical methods of increasing Froude's Number is to increase the nominal tank velocity, it follows that higher values of Reynolds' Number may be expected if efforts are made to increase stability in settling tanks. It should not be lost sight of in this connection that an increase in velocity in a tank of fixed dimensions increases the zone of influence of the inlets and outlets. On the other hand, since removal of suspended matter is more nearly a function of detention period than of tank surface area for most suspensions, an increase in velocity will usually carry with it an increase in length of tank.

In the effort to attain stability by increase of velocity, there must come a velocity beyond which the flow is too turbulent to permit of efficient sedimentation. The Reynolds' Number in terms of the hydraulic radius is a fair measure of turbulence for open channels, and from somewhat meager data on this subject it seems that hydraulic turbulence may start at values of Reynolds' Number between about 300 and 2000.[11] The change from laminar to turbulent flow probably does not produce a sharp change in the settling of suspended particles. In Jukov's [4] work on paper mill wastes, half of the particles of which settled at velocities less than 1.1 mm. per second, there seemed to be very little influence of Reynolds' Number on efficiency of removal until its value reached about 4000. It is probable that most of his runs were with turbulent flow.

The value of Reynolds' Number for a settling tank 20 ft. wide by 15 ft. deep, having a nominal velocity of one foot per minute, is about 8000. It is most likely that the flow in such a tank is turbulent. The velocity of turbulent exchange, which acts perpendicular to the axial velocity, is, however, very small for such a large channel and probably is negligible in comparison with the settling velocities for the example cited. This example is, of course, not unusual. Many existing settling tanks in which clarification is reasonably efficient have higher values of Reynolds' Number. In a settling tank it is probable that currents due to differences in weight, that is, due to gravity forces, are so large as to mask the turbulent exchange.

Conclusions

In conclusion it appears that:

1. A necessary preliminary to the development of a rational method of settling tank design is the understanding and control of the factors which influence settling tank stability.

[10] L. G. Straub, "Transportation of Sediment in Suspension," *Civil Engineering*, May, 1936.
[11] Paper 17 of the U. S. Waterways Exp. Station, Vicksburg, Miss., January, 1935.

2. If a tank is designed and operated so that flow is reasonably stable it is possible to predict removal from quiescent settling analyses of the suspension.

3. A large amount of experimental work remains to be done before correlations can be made between removals predicted in quiescent settling tests and removal actually accomplished in full-scale settling tanks.

4. Once such correlations are made, the operation of model or pilot experimental tanks preliminary to the design of treatment plants should be unnecessary except for unusual cases.

MICROSCOPIC METHODS USED IN BIOLOGICAL INVESTIGATION OF LAKE AND STREAM POLLUTION, AND THE INTERPRETATION OF RESULTS

By Theodore A. Olson

Associate Biologist, Division of Sanitation, Minnesota Department of Health, Minneapolis

Microscopic methods discussed here are concerned with biological investigations made in connection with lake and stream pollution, surveys and closely related problems. The term here is broadened for convenience to include not only those examinations involving the study of very minute forms, such as planktonts, but also those concerned with microscopic bottom-dwelling organisms, such as aquatic insect larvae, Oligochaete worms, leeches, etc., whose positive identification usually involves microscopic examination.

It is a well established fact that organisms occur in, or are absent from, a given locality in proportion to the extent to which the surroundings are favorable or unfavorable to them. Each species or group of species has its individual set of requirements, which, if satisfied, permit success in growth and reproduction.

The existence of a definite relationship between the environment and the individual organisms living in it may be of value in many ways. For instance, when one is armed with the knowledge of physical, chemical, and biological factors constituting an environment, the kind of organisms which may occur can be predicted, and conversely, if this knowledge be lacking and the kind and requirements of the individual organisms which occur are known, then the nature of the environment may be evaluated. It is this latter method which is used in water pollution studies to determine the extent of contamination which exists in a given locality.

The addition of sewage, industrial wastes or other similar substances to water is by general agreement said to "pollute" it, and organisms which can endure and exist under the changed conditions brought about by this pollution are logically called "pollutional forms." Those forms which occur naturally in the stream but disappear when the polluting materials are added are termed "clean-water forms" in keeping with the same system of nomenclature.

To facilitate comparison of the various environments and to permit more accurate determination of the degree of pollution which exists, quantitative methods of investigation are necessary. Standardization of equipment, methods, and expression of results are also much to be desired, if the results obtained by various investigators are to be comparable.

It is desirable at this point to mention again that microscopic methods as here discussed include the consideration of bottom-dwelling forms as well as plankton organisms. The investigational work involved in a biological survey of the extent of pollution in a body of

water may, therefore, he conveniently divided into a study of that free-floating assemblage of organisms known as the plankton and the study of bottom-dwelling forms which are ordinarily larger than the planktonts and which reflect the condition of the supernatant water largely through their reaction to the sedimentary deposit.

In order to attain the greatest accuracy, it is desirable to examine as great a volume of water and as large an area of bottom sediment as is reasonable in expenditure of time and equipment. If some means exists whereby organisms may be removed from a fairly large sample of the water and mud in such a manner that they are all concentrated within a small volume or area, the results obtained will be more reliable than if a small unconcentrated sample is examined. Some cases occur where growths are so luxuriant that organisms may be examined directly, but these are exceptional.

The recommended procedure for plankton concentration is the familiar Sedgwick-Rafter method, involving sand filtration, washing of the sand, and decantation. Compared to a newer innovation, the continous electric centrifuge developed in Wisconsin some years ago, this method is slow and expensive. The centrifuge just mentioned operates at a speed of 20,000 to 25,000 r. p. m. and in operating principle it resembles a "Milk Clarifier." The sloping wall of the centrifuge bowl retains the organisms and approximately 5 c. c. of liquid, while the remainder of the water is thrown out of the machine.

The organisms may be easily removed from the bowl for chemical analysis, enumeration or other means of measurement. When the machine is properly set up, the personal element is reduced to a minimum and the only manipulation necessary is the removal of the catch from the bowl and subsequent dilution to a consistency suitable for microscopical examination. The efficiency of the machine is high and since it may easily be cleaned there is little opportunity for organisms to be carried over from one sample to another. Juday stated that with the exception of *Aphanizomenon*, the first centrifuging removes approximately 98 per cent of the plankton organisms, and as demonstrated by plate counts, 40 to 70 per cent of the bacteria. A slight change in the technique of handling the sample, such as a second centrifuging or skimming the samples before centrifuging will take care of samples containing *Aphanizomenon*.

In using the centrifuge method, it has been found that the time required to concentrate the sample and transfer the "catch" to a sample bottle is roughly 20 minutes for each 2 liters of the original sample. The Sedgwick-Rafter filtration method may require 30 minutes for the filtration of only 500 c. c., and to this should be added the time required to decant the concentrate and make it up to volume. If a large battery of Sedgwick-Rafter funnels is used to speed up filtration, the cost of equipment mounts rapidly and much valuable laboratory space is taken over.

There are also other means of concentration which have good points. One of these is a simple inexpensive method which relies on a long

period of sedimentation following preservation. The sample is allowed to stand for a period of at least two weeks after preservation, entirely undisturbed. At the end of this time a siphon tube, the inlet end of which has been covered with a piece of No. 25 bolting cloth, is carefully lowered to the middle of the sample bottle and the siphon is started. The volume is thus reduced to the point where there is danger of disturbing settled organisms. The method is inexpensive with reference to the equipment needed; however, it can be very inaccurate in inexperienced or careless hands, and the final volume is usually greater than that obtained by other methods.

Concentration by the use of a hard filter paper is another inexpensive method, but is often very slow and some organisms become imbedded in the filter paper to a considerable extent.

This brief discussion does not include all possible means of concentration of plankton samples, but indicates a few of the advantages and disadvantages encountered in some of the common procedures. It seems apparent that the centrifuge method mentioned has a sufficient number of good points to be considered favorably, and that it should have a place as prominent as the Sedgwick-Rafter method in acceptable laboratory technique.

It is sometimes desirable to give special attention to larger plankton forms, such as *Cladocera, Copepoda, Rotifera* and the like. Since these occur less abundantly than the smaller plankton forms, a larger original sample is desirable. Only exceptionally are the larger forms numerous enough to insure an accurate count in a plankton sample having a volume of a liter or two. It is suggested, therefore, that the regular plankton sample, collected and concentrated in the usual manner, should be supplemented by the collection of a 100 liter sample which is concentrated by straining through a silk net. Silk bolting cloth having approximately 40,000 meshes per square inch should be used in the manufacture of the net.

With reference to the collection of samples, any method which obtains a representative dispersion of organism and prevents undue contamination with extraneous material should be suitable. The importance of immediate preservation should however be emphasized. Many forms disintegrate rapidly and the immediate addition of formalin to the collected sample is advisable. When the examination of a fresh sample containing living motile organisms is necessary, the actual examination should be prompt and the sample should be kept as cool as possible.

The study of the bottom fauna involves an entirely different procedure than the plankton examinations. First of all, it is more difficult to obtain a quantitative sample, and for this reason a specially constructed dredge must be used. At present there are two types which have received favorable comment and are much used in fresh water aquatic work. One is known as the Ekman dredge and the other as the Petersen dredge, each being named after the designer of the original marine type. These dredges are made in a variety of sizes and weights,

Flexibility relative to the weight of the apparatus is often desirable in order to facilitate sample taking in all types of bottom sediment, but for uniformity of results a standardization of the amount of sample taken by each is of considerable importance. In other words, it would be desirable to have dredges which may be varied as to weight, but which are all alike with reference to the surface area from which they collect a sample. Furthermore, all results obtained should be reported in a standardized manner. This latter condition must be observed if ready comparisons are to be made either in connection with the various projects of one individual investigator or in connection with the scattered work of several investigators.

Inasmuch as the collection of samples and the manipulation of the material after collection are of paramount importance, these phases of the investigation should receive some attention. The selection of a sampling point which best represents the condition prevailing is also very necessary. Small, usually insignificant, areas entirely unrepresentative of the rest of the surroundings may exist within an environment. A fairly large spring flowing into a heavily polluted stream would represent such a microenvironment. In a case on record a large trout lived within the zone of influence of such a spring. Any attempt on the part of the fish to leave the area would have placed it in danger of suffocation from lack of oxygen in the polluted water and its survival in the surrounding water should perhaps have been a matter of minutes only. It is evident that the general condition of the stream could not in fairness be judged from an examination of the fauna in the area inhabited by the trout. Likewise it would be inadvisable to use results obtained through a study of samples taken very near the bank. Stream velocity is reduced near the edge of the stream and shallows exist where conditions are usually quite different than in the channel itself or in the deeper water of the eddies and pools. However, each locality offers a new problem, and good technique is attained principally through the utilization of common sense in association with a measure of experience.

The treatment of the sample after it has been collected by the dredge may be varied. However, the following procedure is offered as one which has been used successfully over a period of several years of aquatic work in Minnesota. The sample as collected usually contains much fine debris and silt which makes preservation difficult, and for this reason each sample is washed clean of this material in the field before it is preserved and taken to the laboratory. Ordinarily, a considerable amount of water is included in the sample as it is taken. The first step is to stir the material vigorously, mixing the water with the sediment, after which it is allowed to stand for a moment. The supernatant liquid is then strained through a 30 mesh and 100 mesh sieve successively.* The concentrate thus obtained is preserved with formalin and forms a supplement to the regular sample. It contains minute forms which escape the coarse meshes of the ordinary 30 mesh screen. The remainder of the sample is washed with the aid of the coarse sieve

* Sieves having 30 meshes and 100 meshes to the inch.

and sand, organic debris, organisms and other material which are held back are preserved by the addition of formalin.

Experience has shown that the next step, that is, the removal of organisms from the concentrated sample, is more readily accomplished when the material has been preserved than when the material is fresh. Since each organism must be removed by hand with the aid of a pair of forceps, the lack of swimming or creeping movements facilitates the removal of organisms in a preserved sample. This same lack of movement and an inability to adhere tightly to objects means that organisms may be washed out of the sample rather readily, and for this reason a second washing is given the sample at the laboratory as a further means of concentration. Formalin fumes which are irritating to mucous membranes of the nose and throat, are also removed in this operation.

A strong stream of water is directed into a pail containing the sample in such a manner that the material is well agitated. When the water is shut off, coarse sand and heavy debris immediately settle out and the organisms which are lighter may be poured off with the supernatant water. By repeating this washing operation several times and by using a 30-mesh sieve to strain the water, almost all of the organisms occurring in the sample will finally be concentrated on the sieve. From this small sample the organisms may readily be removed for enumeration and identification. The examination of the heavier sediment which remains in the sample pail after the washing process may be accomplished fairly rapidly, inasmuch as very few organisms remain in it.

When an interpretation of results is to be made, the first step required is a classification of the organisms into groups according to their reaction to pollution. This is not always easily accomplished. The system now in use which is presented by Fair and Whipple in the "Microscopy of Drinking Water" divides the organisms into four groups. There are two divisions representing different degrees of tolerance to pollution and two divisions representing degrees of intolerance to pollution. The distinction between the degree of pollution and lack of pollution is a relative thing, however, and for that reason it is often hard to distinguish between one degree of pollution and another.

It is with respect to plankton organisms that difficulty is usually experienced in drawing hard and fast lines; bottom-dwelling forms seem to show a greater consistency in their reaction to pollution. Plankton forms, of course, have no efficient swimming movement but are carried along passively by the water currents, and even when conditions are unsuitable, they may exist in the upper portions or the polluted section of a stream merely as a result of the fact that they have been carried in from cleaner water farther upstream.

In bacteriological work, it is not necessary that all bacteria occurring in a sample should be named and classified according to their pollutional index, in order that the information obtained from a sample shall

be of value. In fact, we may take the presence of one species or a small group of species as indicative of pollution.

Applying the same method to studies of the plankton and bottom-dwelling forms, we find that we have several index organisms to choose from. For instance, among the forms living in the sediment of a polluted stream or lake, we will find several species of oligochaete worms which thrive under the adverse conditions accompanying pollution. Certain hardy leeches, insect larvae, and molluscs may also be found in such an environment.

The restrictions imposed by adopting the method of interpretation just mentioned is not necessary, however, for the ecology of the plankton and bottom-dwelling organisms is sufficiently well established to make us independent of a single index organism. It may be suggested here, however, that it may not always be necessary to make exceedingly fine distinctions. Organisms could be listed as pollutional forms, clean-water forms, and facultative pollutional forms. Mesosaprobic and polysaprobic forms would thus be grouped together as pollutional organisms and oligosaprobic and katarobic forms as clean-water forms. The facultative pollutional group would include organisms on whose reaction to pollution authorities are disagreed, organisms which are air breathers or in other ways so organized that they are not good indices of pollution, and, finally, if so desired, organisms whose reaction to pollution has not yet been observed. In most cases the latter are not numerous, but they should be accounted for in some way. The majority of organisms occurring in the facultative pollutional group prefers clean water which is well aerated, but they can also endure the hardships imposed by moderate pollution and are therefore found living in such environments. This may explain the fact that authorities have variously described them both as mesosaprobic and oligosaprobic in their reactions. The air breathing forms, if omnivorous in their feeding habits, may be equally abundant in either polluted or clean water.

The clean water environment may have a few pollutional forms, but the majority of the organisms will be found to belong to the facultative pollutional and clean water groups.

As contaminating material is contributed, the pollutional forms and often times the facultative pollutional group increase slightly in numerical value, while the more sensitive form begin to decrease. As the water becomes more and more polluted, it becomes progressively more unfit for a wide variety of forms. Clean-water forms disappear first and are soon followed by the facultative pollutional forms. This reduction continues until an extreme is reached where only one or two of the most tolerant species remain. However, even where serious pollution exists there may be an enormous number of individual specimens of the hardy species which do occur.

There is no need to proceed beyond the explanation of the system of groupings just mentioned, for the procedure from that point on is fairly familiar to every one. However, as a last thought, it may be suggested that a consideration of the plankton or the bottom organisms

as groups of organisms or communities might be substituted for the consideration of the individual index organism. Under this system it might be possible to classify a community under a number of terms each indicating the existing degree of pollution.

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COMPARISON OF SEWAGE PURIFICATION BY COMPRESSED AIR AND MECHANICALLY AERATED ACTIVATED SLUDGE *

By G. M. Ridenour and C. N. Henderson

I. Purification and Sludge Settling Characteristics

Research Engineer and Laboratory Assistant, Division Water and Sewage Research

With the increased interest in the activated sludge process of sewage treatment in recent years, the relative merit of compressed air versus mechanical aeration has been the subject of much discussion. A perusal of the literature as far back as 1918 shows, however, that evidence is yet inconclusive as to the relative advantages or disadvantages of these two types of aeration.

Available reports show that proponents of each type are more or less equally divided. Such investigators as Hayworth,[9] Bolton,[2] Buswell,[3, 4] Peck,[13] Butler and Coste,[6] Buswell, Shive and Neave,[5] and Kessener and Ribbius[10] conclude that mechanical aeration involving surface absorption of oxygen is the more efficient process. Others, Sandford,[14] Dallyn and Delaporte,[7] Fowler and Chatterjee,[8] Nordell,[12] and Scouller and Watson[15] hold the view that compressed air is the better type of aeration. Still others, Sierp,[16] Makepeace,[11] and Besselievre[1] believe that a combination of the two types is the best.

This lack of agreement may be expected because of the complexity of the process and the difficulty of securing dependable data of a comparable nature. Accurate comparison of the relative efficiency of the two types of aeration involves a number of factors which must be held constant and it is difficult to meet this requirement except under very unusual circumstances. The conclusions that have been reached appear to have been based largely on a comparison of results from various separate plants. Under these conditions the effect of local variable factors such as quality and quantity of sewage, structural and hydraulic design and method of operation can easily obscure results. The equality of such factors as character of sewage, flow rates, quality and quantity of return sludge, dimensions of units, hydraulic features and methods of plant operation are of greatest importance in comparing aeration systems.

A few investigations have also been made on laboratory scale under controlled conditions. These investigations have furnished excellent indications, but it is doubtful whether these laboratory methods have allowed the full weight of operating conditions to be applied.

The writers have had an opportunity in the past few years to study the relative merits of a compressed air and mechanical aeration system under practical operating conditions which were believed to be ideal

* Jour. Series Paper, N. J. Agricultural Experiment Station, New Brunswick, N. J. Division Water and Sewage Research.

as far as comparable circumstances were concerned. The occasion provided simultaneous treatment of sewage by activated sludge in tanks of identical structural and hydraulic design, using the same sewage and working under uniform methods of operation. The only difference in plant design and method of operation was the type of aeration. One system consisted of surface mechanical aeration by means of brushes. The other was compressed air aeration by means of spiral circulation.

The results of these investigations are offered in three sections as follows:

I—Purification and sludge settling characteristics

II—Cost of operation, construction and operation requirements

III—General discussion of applicability of the two types.

Description of Plant Facilities

The plant where these studies were conducted is located at Marlboro, N. J. The designed plant capacity is 1,300,000 gallons per 24 hours. The units consist of a screen chamber, a preliminary settling tank, three aeration units provided with alternative method of aeration in the same tank, three final settling tanks, one chlorination chamber, two sludge digestion tanks, glass covered sludge drying beds, mechanical house and laboratory. A diagrammatic sketch of the layout is shown in Figure 1 and a photographic view of the two aerators are shown in Figure 2.

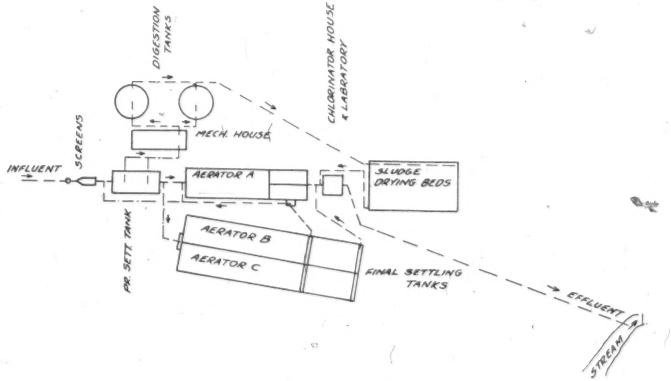

Fig. 1.—Diagrammatic sketch of plant layout.

Aerators.—The aeration tanks, with alternative facilities for aeration, are shown in sketch in Figures 3 and 4 and in actual operation in Figures 5 and 6.

The brush type of aeration was designed for a range of speed between 43 and 84 revolutions per minute with a dip of the brush tips in

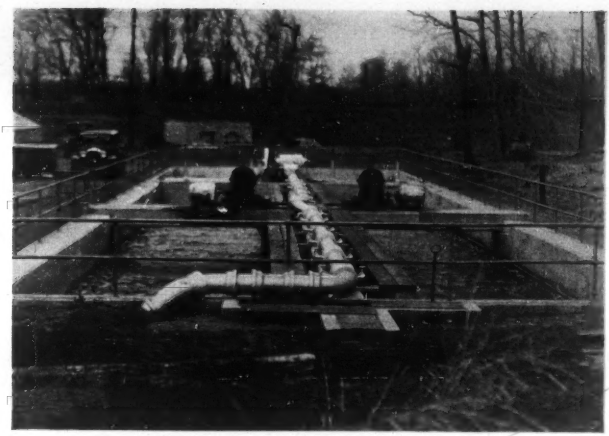

FIG. 2.—View of aerators.

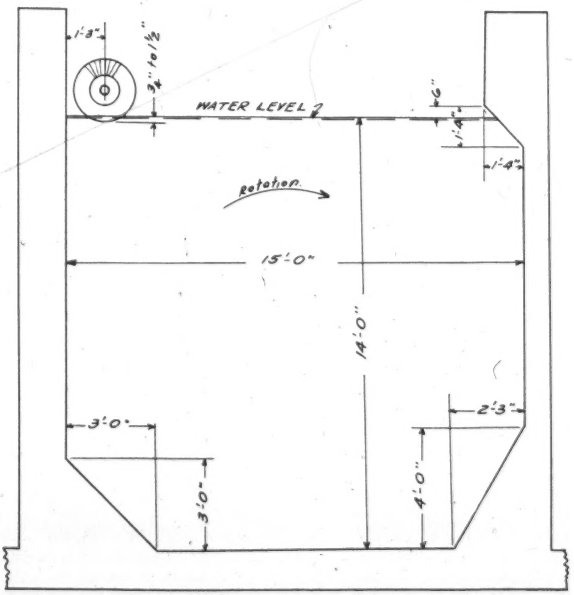

FIG. 3.—Cross-sectional sketch of brush aerator installation.

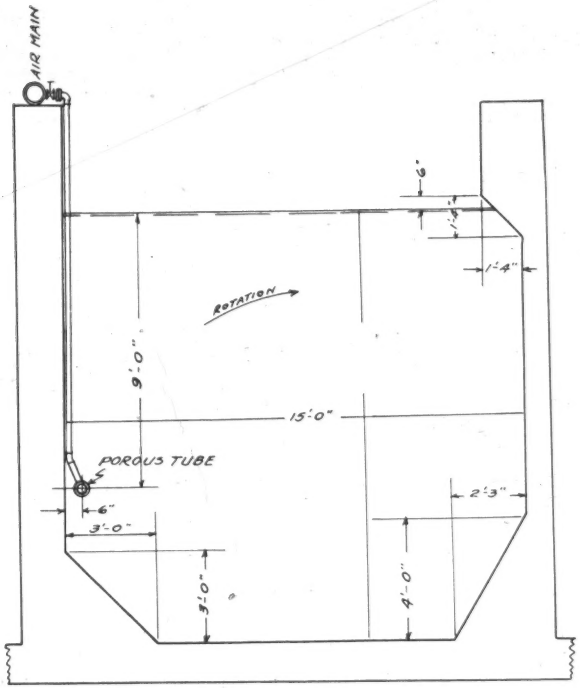

FIG. 4.—Cross-sectional sketch of compressed air installation.

FIG. 5.—View of brush aerator in operation.

FIG. 6.—View of compressed air aerator in operation.

the liquid which could be varied from 0 to 3 in. During these studies the depth was maintained at 1½ in.

The compressed air facilities consisted of duplicate rotary type compressors capable of delivering from 0.5 to 1.5 cu. ft. of air per gallon of sewage per minute. The diffuse tubes, shown in sketch in Figure 7, were Norton "medium porous," with a dry permeability of

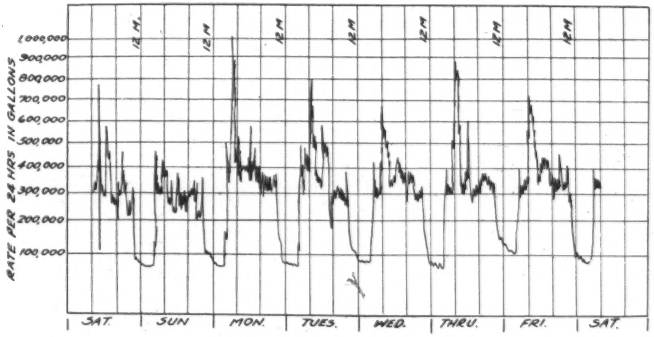

FIG. 7.—Typical sewage flow chart.

20 cu. ft. of air per sq. ft. per min. under 2 in. water pressure. Each foot of tube was capable of passing 14 cu. ft. of air per min. under a maximum of 4 lb. pressure. The tubes were 27 in. long with 3-in. bore and ⅝-in. wall. The depth of 9 ft. below the water level was selected as representing, from practical experience, the minimum for most efficient performance with most economical operation.

Each aeration tank was equipped with cross-baffles in the same position with respect to the length for the purpose of preventing "core" action.

The effluent from each aerator discharged into an individual final settling tank equipped with a longitudinal scraper-type of sludge collecting mechanism, which delivered the settled sludge continuously to hopper-bottom sumps at the end of each final settling tank. The sludge was drawn continuously from the sumps by means of air lift, and discharged into the settled sewage channel leading to the inlet end of the aerators.

Of particular interest to this study was the flexibility of the plant, which permitted the use of a common sewage for all aerators, a common or specific return sludge, and independent use of either brushes or porous tubes for aeration, in sufficient amount to maintain a liberal supply of oxygen in either aeration unit with activated sludge solids as high as 4000 p.p.m.

All structural and dimensional characteristics of the tanks were identical.

Quantity and Type of Sewage.—The average daily sewage flow rate during this investigation varied between a minimum flow of 300,000 to a maximum of 450,000 gal. per 24 hours. During the 24 hr. daily period the rate of flow varied from a low of 80,000 to a peak rate of 1,000,000 gal. per 24 hours. These flow characteristics are represented by the typical weekly flow chart shown in Fig. 7.

The sewage was approximately equivalent to a medium strength domestic sewage, with the exception that the kitchen and laundry waste content were at times in much higher concentrations than ordinarily encountered in such sewages. Small amounts of dye wastes were also present at times. An analysis of a typical 24-hour composite sample is shown in Table I.

TABLE I.—*24-Hour Composite Analysis of Raw and Settled Sewage*

Raw Sewage	
pH =	
Suspended solids	156.0 p.p.m.
Settleable solids	4.0 cc. per liter
Total solids	580.0 p.p.m.
Ether-soluble material	43.3 p.p.m.
Dissolved oxygen	1.0 p.p.m.
B.O.D. (5-day)	235.0 p.p.m.
Total acidity	0.5 p.p.m.
Total alkalinity	130.0 p.p.m.
Chlorides	21.0 p.p.m.
Settled Sewage	
pH = 6.9	
Suspended solids	124.0 p.p.m.
Settleable solids	1.0 cc. per liter
Total solids	498.0 p.p.m.
Dissolved oxygen	0.0 p.p.m.
B.O.D. (5-day)	174.0 p.p.m.
Total acidity	13.0 p.p.m.
Total alkalinity	100.0 p.p.m.
Chlorides	21.0 p.p.m.

METHOD OF INVESTIGATION AND OUTLINE OF RESULTS

Observations on the comparative sewage purification ability and settling characteristics of the activated sludge floc formed by the two types of aeration were made by two separate methods of approach.

First, a study was made of the relative purification and settling characteristics of the activated sludge floc for each type of aeration, with the use of a common return sludge. Second, the same characteristics were studied using a return activated sludge specific for each type of aeration. Under the first method, the mixture of activated sludge formed by compressed air and brushes was equally distributed as a return sludge between the compressed air and brush tanks, operating

TABLE II.—*Average Purification Results at Outlet End of Aerator*
With Use of Common Return Sludge

	Sewage			Liquor			Activated Sludge		
	B.O.D.	D.O.	NH_3–N	NO_2–N	NO_3–N	Suspended Solids	Suspended Solids	Ash %	Ratio
Settled Sewage	174.0	0	12.7	0	0	124.0			
Tank C—Air	6.0	2.64	5.88	0.031	9.05	8.1	1809	24.06	4.91
Per cent Reduction	96.8		53.7			93.4			
Tank B—Brushes	6.32	3.44	6.00	0.026	8.86	16.3	1900	23.50	4.65
Per cent Reduction	96.3		52.8			86.8			

TABLE III.—*Average Purification Results at Outlet End of Aerator*
With Use of Specific Return Sludge

	B.O.D.	D.O.	NH_3–N	NO_2–N	NO_3–N	Suspended Solids	Activated Solids	Ash %	Ratio
Settled Sewage	182.0	0.0	14.0	0.0	0.0	134.0			
Tank C—Air	1.9	1.4	4.1		1.3	5.1	2460	35.1	7.7
Per cent Reduction	99.0		71.6			96.2			
Tank B—Brushes	3.5	1.6	6.0		1.1	16.0	2270	30.4	2.5
Per cent Reduction	97.4		57.0			88.0			

TABLE IV.—*Progressive Purification Results*
With Use of Common Return Sludge

	Sewage Sludge						Activated Sludge		
	B.O.D. p.p.m.	D.O. p.p.m.	NH_3–N p.p.m.	NO_2–N p.p.m.	NO_3–N p.p.m.	Suspended Solids p.p.m.	Total p.p.m.	Ash %	Ratio
Point No. 1 (5 minutes)									
Air	12.3	0.97	1.90	0.58	11.3	16.2	2290	29.9	8.87
Brushes	14.7	1.39	1.40	0.74	14.3	22.0	2249	30.0	9.30
Point No. 2 (11 hours)									
Air	6.6	2.5	0.07	0.19	13.0	10.2	2211	30.2	9.30
Brushes	7.0	3.7	0.14	0.14	16.7	18.0	2235	30.4	9.1
Point No. 3 (22 hours)									
Air	5.2	3.87	0.00	0.024	16.7	7.8	2194	30.4	9.33
Brushes	4.0	5.3	0.04	0.043	17.0	16.2	2270	30.4	9.07

in parallel. Under the second method, the return activated sludge formed by each type of aeration was returned to the aerator in which it was formed.

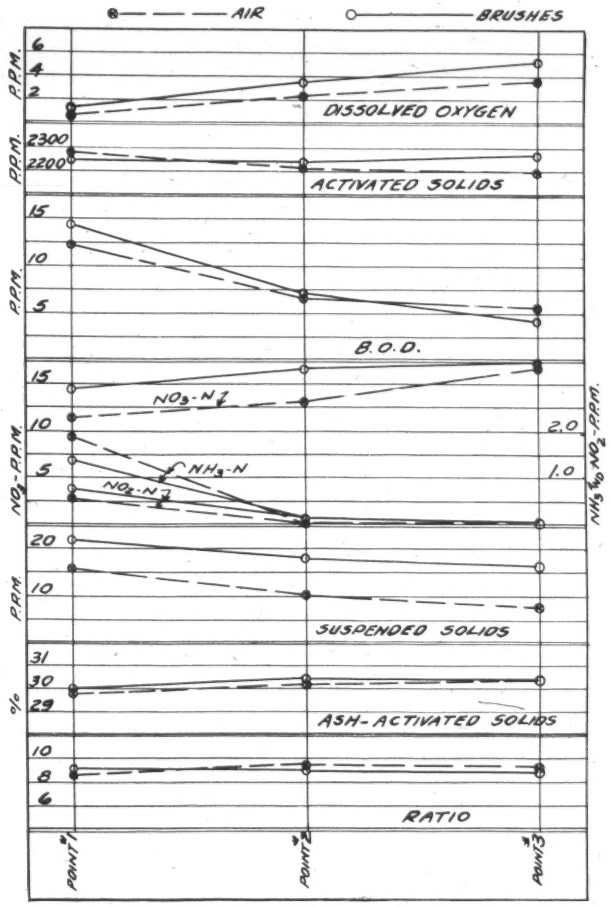

FIG. 8.—Progressive purification results in aerators.

The two methods of approach were considered necessary to secure complete elimination of all variable factors. With the use of a common return sludge all variables were entirely eliminated but under such operation it was believed that the sludge floc would not develop its full characteristics, especially if the two types of aeration should show divergent characteristics of purification or sludge settling. On the other hand the use of sludge specifically developed from each type of aeration might permit one slight variation from an otherwise absolutely

comparable condition. This might occur by reason of slight differences in handling the return activated sludge under the latter method. The two separate methods of approach obviated this probability by serving as a cross-check on the other.

Indices used for purification in these studies were B.O.D., suspended solids and ammonia-N reduction, and nitrite-N and nitrate-N formation. These tests were made on the supernatant liquor at intermediate points in the aerator as well as on the final effluent. Dissolved oxygen tests were conducted on the supernatant liquor of the activated sludge mixture after 30 min. settling.

The sludge settling characteristics were expressed by the ratio of suspended solids in p.p.m. to the volume occupied by the sludge after 30 min. settling in a one-liter cylinder.

The numerical results of the purification and sludge settling characteristics of each type of aeration are shown in the accompanying tables and graphs. Table II shows the average purification and sludge settling characteristics obtained with a common return sludge. Table III shows similar average indices obtained by using a return sludge specific for each type of aeration. Table IV and Figure 8 show the progressive purification results obtained through the aerator at the beginning, middle and ends of the tanks. Table V and Figure 9 shows the results secured at the outlet end of the aerator with the use of varying amounts of activated suspended solids in the aeration tanks.

The results shown were obtained between June, 1933 and April, 1935. The average values shown in the tables are compiled from approximately 88 complete weekly analyses and 600 partial daily analyses.

TABLE V.—*Results at Point No. 3 with Varying Quantities of Solids*
With Use of Common Return Sludge

	Sewage Liquor					Activated Sludge	
	B.O.D. p.p.m.	NH_3 p.p.m.	NO_2 p.p.m.	NO_3 p.p.m.	Suspended Solids p.p.m.	Ash %	Ratio
Activated Solids 1183 to 1300 p.p.m.							
Brushes	7.0	trace		14.0	29.1	29.0	13.7
Air	9.0	trace		13.0	15.8	28.9	12.0
Activated Solids 1700 to 1800 p.p.m.							
Brushes	2.8	trace		14.1	20.0	29.4	10.2
Air	3.0	trace		13.0	8.6	28.8	10.5
Activated Solids 2150 to 2250 p.p.m.							
Brushes	3.3	trace		18.0	13.6	30.0	10.1
Air	2.5	trace		16.0	6.8	30.1	10.2
Activated Solids 2950 to 3150 p.p.m.							
Brushes	1.0	0		19.8	2.0	29.0	5.2
Air	1.0	0		21.5	1.4	28.7	5.7

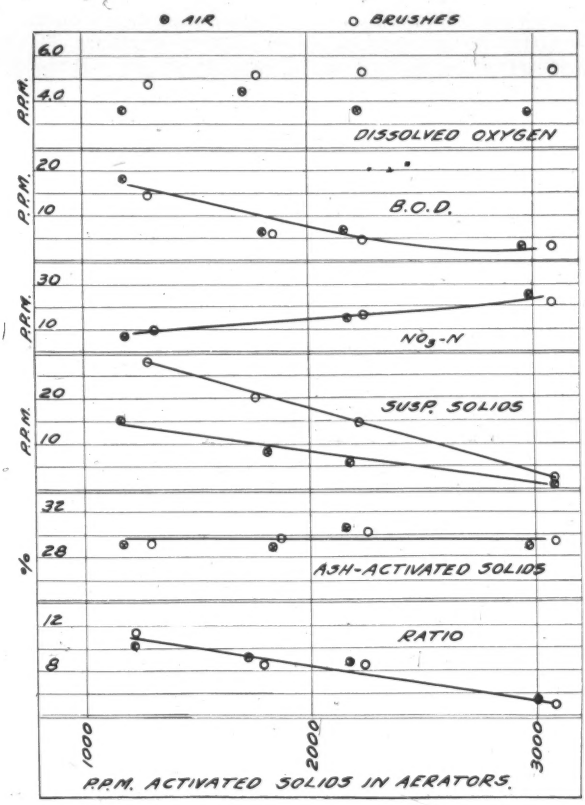

FIG. 9.—Purification with varying amounts of activated sludge.

DISCUSSION

The results may be interpreted most clearly by considering them from two separate viewpoints; first, for the comparative purification performance and second for the comparative sludge settling characteristics.

Purification.—From the standpoint of purification the results permit a comparison of the two aerators in four respects, *i.e.*, (*a*) using a common return sludge (*b*) using a return sludge specific to each type of aeration, (*c*) purification at progressive points in the aerator representing aeration periods of 5 minutes, 11 hours and 22 hours, respectively, and (*d*) purification with the use of varying quantities of activated solids in the aeration tanks.

These data as a whole show a marked equality in purification in the two aerators with one common and consistent exception. This exception is the difference in the removal of suspended solids through the aerators. The extent of this difference varied with the quantity of activated solids carried in the aerators. On the basis of average purification, with the use of a common return sludge, the effluent from the brush aerator contained 16.3 p.p.m. suspended solids while the compressed air tank effluent contained only 8.1 p.p.m., or an overall removal efficiency through the aeration tanks of 86.8 and 93.4 per cent, respectively. With the use of a specific return sludge the same characteristic difference existed as with the common return sludge. Although actual values were not alike the difference in removal efficiency was almost exactly the same, or 6.2 per cent. In the data showing purification at progressive stages through the aerators, the differences in suspended solids removal are shown even more clearly, not only by the similar differences in suspended solids removal, but also by the lesser rate of suspended solids removal through the brush aerator. The results obtained with varying quantities of activated sludge solids, while showing a similar difference in purification characteristics brought out, however a fact not previously observed. The actual differences in suspended solids removal, varied with the quantities of activated solids used. At and above 3000 p.p.m. activated solids the difference in suspended solids removal was practically negligible. But decreasing from 300 p.p.m. the difference in removal varied almost as a straight line function to the lowest quantity studied.

With respect to other purification indices the two aerators gave quite similar results. Such differences that did exist are considered to be within the limits of experimental errors of methods of analyses or sampling. In connection with these other indices of purification, there is one point of evidence which is of significance when related to the differences in suspended solids removal previously discussed. It will be observed that the differences in suspended solids removal did not carry a corresponding difference in B.O.D. removal, as might be expected. Taking for example the data given in Table II, the suspended solids in the effluent from the brushes and compressed air tanks were 16.3 and 8.1 p.p.m. respectively. However, the average B.O.D.'s of the corresponding samples were 6.32 and 6.00 or a difference of only a fraction of a part per million. This point is of significant importance from the standpoint of practical application of the two aerators.

Sludge Settling Characteristics.—With respect to settling characteristics of the sludges formed with the two types of aeration, Tables II to V and Figures 8 and 9 show a comparison on the same basis as previously used for purification results, namely (*a*) with the use of a common return sludge, (*b*) with the use of a return sludge specific to each type of aeration, (*c*) at progressive points in the aerator, and (*d*) with the use of varying quantities of activated sludge solids in the aerators.

The combined results show that compressed air gave a definitely

heavier and denser sludge floc than did the brush type of aeration. This fact is given most clearly in Table III, where a specific return sludge was used for each aerator. The settling and compacting ratio of the sludge floc formed by the brushes under these conditions was 2.5 as against the ratio of 7.7 for the compressed air, for the same period. This represents a density of sludge floc by compressed air of more than three times that formed by the brushes. All the other results, although not having been permitted to develop their full differential values, due to the conditions of operation, show the same tendency for formation of heavier floc from the compressed air tank. The average yearly results with the use of a common return sludge, given in Table II, indicate a slightly better settling sludge from the compressed air tank. In Table IV, which shows progressive purification results with the use of a common return sludge, the compressed air floc showed a tendency to become heavier with increasing time of aeration, whereas the sludge in the brush aerator became increasingly lighter at corresponding points. In Table V, showing the comparative ratios with varying quantities of activated sludge solids with a common return sludge, the compressed air gave a slightly higher settling ratio than the brushes, with the exception of the lowest quantity studied. The results from this lowest quantity, with reference to purification, should probably be excluded because of increased importance of other factors.

Combined Results.—The combined results presented thus far would show therefore that the two types of aeration differed in only two points of significance: (*a*) less efficient removal of suspended solids by the brush aerators and (*b*) the formation of a less dense sludge floc by the brush aerators.

The first point of divergence is probably not of very great practical significance, because of the fact that a corresponding difference in B.O.D. reduction through the two aerators did not exist. This failure of B.O.D. reduction to accompany the difference in suspended solids removal was apparently because of the highly oxidized state of the excess suspended solids which existed in the brush aerator effluent. All other conditions of the effluent being approximately equal, especially with respect to diss. oxygen, B.O.D., ammonia-N, nitrite-N and nitrate-N, it would perhaps be unfair to form an opinion on the comparative merits of the two types of aeration solely on the basis of this small difference in suspended solids removal. The difference is of even less significance when it is considered that the greatest differences existed in the lowest activated solids ranges, which are below those customarily employed.

The greater quantities of suspended solids in the brush aerator effluent, by virtue of their highly oxidized state, are probably of even still lesser significance with respect to their effect on the receiving stream.

The differences in performance of the two aerators, with respect to the density of the sludge floc formed, would however be of major practical importance. A difference of 300 per cent in settling and com-

pacting ability of the sludge floc makes an important difference in sludge handling requirements. This difference would be directly effective in altering capacities of the final settling tanks, sludge return facilities, concentration tanks, sludge digestion tanks, and sludge drying beds, to handle the lighter sludge.

If these additional capacities were not provided, the maximum amounts of activated sludge solids which could be carried in the aeration tanks would be considerably lower for the brush aerators than for the compressed air.

This reduction in activated solids in the aerators would be effective in permitting the use of smaller capacities in the final settling tanks, but the requirement for additional capacities in the digestors would still remain approximately the same, since after equilibrium of operation was established the quantities of solids to be wasted daily would be equal in each case. Reduction in quantities of solids would also be possible only in limited situations where sewage strength was either less or else not subject to extreme fluctuations in organic load.

It seems therefore from the practical standpoint, that the significant differences in the two types of aeration, with respect to purification performance and sludge settling characteristics as affecting plant design or operation, are limited to the difference in density and volume of the activated sludge floc which is formed. This difference, without consideration to other factors (such as operating cost versus cost of capital construction for each type of aeration) would be sufficient to favor compressed air. If, however, differences in operating costs of the brushes should be sufficiently great to more than offset the additional capital costs for handling the lighter sludge from the mechanical type of aeration, the weight in favor of compressed air would be lost. Investigations on comparative costs of operation are given in the next installment.

Summary

Investigations were conducted on an activated sludge plant, under practical operating conditions, to determine the relative merits of aerating activated sludge—sewage mixtures by means of mechanical surface aeration and compressed air.

The method of investigation consisted of comparing the purification and sludge-floc settling characteristics of the two types of aeration under identical conditions of tank design, operating requirements and method of operation. The type of aeration was the only variable factor.

The results of the investigation, with respect to the comparative purification and sludge floc settling characteristics, have led us to the following conclusions:

(*a*) From the standpoint of purification of the sewage, the two aerators produced almost identical results with the use of 3000 p.p.m. or more activated sludge solids in the aerators. Below 3000 p.p.m., however, the brushes removed less suspended solids than the compressed air. This difference in removal increased readily, as a straight

line function, with decreasing amounts of activated solids. The differences in suspended solids removal failed to create any difference in the B.O.D. removal efficiencies.

(*b*) The difference of most practical significance in the performance of the two aerators was in the settling and compacting ability of the sludge floc. The settling and compacting ability of the sludge formed by the compressed air tanks was three times as great as that of the sludge formed by the brushes.

Results of the study with respect to comparative costs of the two systems and applications are to be presented in following sections.

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Plant Operation

NEW YORK STATE ASSOCIATION CONTEST FOR EXCELLENCE OF TREATMENT PLANT OPERATING REPORTS

FOREWORD

By C. A. Emerson, Jr.

Chairman, Federation of Sewage Works Associations

The award of a loving cup to the operator adjudged as having submitted the best annual report on treatment plant operation in the contest held by the Rating Committee of the New York State Association should be a matter of interest to every reader of this *Journal*.

Through the friendly rivalry of a contest the New York Association is striving to secure state-wide adoption of a uniform system of reports of operation of sewage treatment plants and likewise to improve the accuracy and completeness of the reports. The object of the contest is certainly praiseworthy and both the Committee and the Association are to be congratulated on the outcome of this first year's work.

The uniform excellence of all the reports submitted by the comparatively large number of contestants the first year was gratifying and together with the fact that more entries have been received for this year's contest seems conclusive proof that adequate record keeping is not the almost impossible task some claim it to be and one which requires the addition of a bookkeeper or trained accountant to the operating force.

The records required by the New York Committee (*Journal*, March, 1935) are quite similar to those recommended by the Federation Committee on Sewage Works Operating and Cost Records under the Chairmanship of C. E. Keefer (*Journal*, January, 1932), but the scheme of rating is new and permits the operator of a small and simple plant to enter a contest on an equal basis with the operator of a large complex plant.

The award was based entirely upon the adequacy of operating records which is a matter wholly within the province of the operator and did not give consideration to the quality of performance of the plant which might be affected by overloading or other causes quite outside the operator's control.

If the number of entries can be increased from year to year and similar steps can be taken by other associations it will only be a short time when a mass of operating data will be available which will be of unquestioned value for comparison purposes in judging the quality of performance of an existing plant or as a guide in design of a new installation or an extension. The lack of such data has long been keenly felt and any method which has reasonable promise of filling this need is worthy of widespread support.

The officers of the Federation were pleased to foster this movement in New York State and will be glad to do likewise for any other associations. Not the least valuable assistance which the Federation can render will be the publication of summaries of these operating reports in the Operators' Reports and Suggestions Department of this *Journal* so they are available to all for information and comparison purposes.

REMARKS ON PRESENTATION OF RATING COMMITTEE AWARD, ALBANY, NEW YORK, JUNE 11–13, 1936

By Anselmo F. Dappert

Chairman of the New York State Sewage Works Association Rating Committee and Principal Sanitary Engineer, New York State Department of Health

Here displayed before us is the highest token of esteem and respect which this Association is able to bestow upon one of its operator members. Being the first of its kind it is fitting that we reflect a little upon its history and its significance. This honorary award which I am about to present in behalf of the Association is to be given in recognition of the high quality and excellence of a report submitted to the Rating Committee by one of our operators, covering twelve months of operation of his plant.

This ceremony marks the culmination of the first stage of a movement begun at our meeting in Oneonta in October, 1934, when a committee was appointed to recommend a system of ratings for sewage treatment plants based on the adequacy of operating data and records, irrespective of size of plant or type of treatment. As you know, the Committee made its report at our January meeting in 1935 and recommended a system for rating which was subsequently adopted, the report and outline of which has been widely distributed. The Committee in formulating the rating system faithfully carried out the mandates of the resolution which created it. The system, as devised, establishes an annual report on operation as the basis of rating and places a premium on operating data rather than the results of treatment or the equipment and plant facilities. The system is fair and equitable to all operators alike regardless of the type or size of plant. It gives the operator of a small simple plant equal if not more favorable opportunity for achieving a high rating than the operator of a large complex plant. It rewards the operator for the application of individual effort and initiative, which are things within his control.

The philosophy back of the plan, of course, is to raise the standards of plant operation, which will be accomplished when the operator fully understands the functions of the units of his plant and keeps a set of books on their accomplishments. It has the further purpose of providing the means whereby the results at one plant may be compared with those of another and thus contribute to advances in the art and practices of sewage treatment, which invariably follow the interchange of information based upon wide operating experiences. The plan in no way attempts to set up a competitive system of gradation of the qualities of operators. It affords only the opportunity to each operator of acquiring some honor and recognition for his services and for energies applied largely outside of his working hours, and the opportunity of making a definite individual contribution to the advancement of the profession. It is a plan which I hope to see extended among all

operators throughout the country, the advantages and benefits of which I am sure are now apparent to the eleven New York State operators who co-operated in making this venture a success in its first year by submitting reports to the Rating Committee. I am confident that, as a result of his participation in the enterprise, each of these operators knows a little more about his plant now and has discovered some new interests which formerly did not exist for him.

For the ten operators who submitted reports, but who this year will not be the recipients of this beautiful award, let me say that their co-operation has nevertheless been greatly appreciated. There is honor for them in the fact that they entered into the spirit of the project and have displayed a desire to lead the way to new achievements in the field. Their reports on the whole were gratifying. But the rating system is rather inflexible—there can be only one winner—and credit cannot be given unless the information is presented for the numerous items called for in the outline. These ten operators should in no way feel discouraged by their failure to be selected for the award. They should and undoubtedly will feel disposed, along with many others who did not submit reports this year, to come back again next year, and with the benefit of this year's experience, be able to achieve a higher score. I think it is particularly desirable that public recognition be given to these ten operators who submitted reports and must be satisfied with the contribution which they have made to the success of the Rating Plan. They are Mr. Haemmerlein of East Aurora, Mr. Cary of Fairport, Mr. Skinner of Monticello, Mr. Mathers of Garden City, Mr. McBreen of Rockland State Hospital, Mr. Strandburg of Jamestown, Mr. Sampson of Tonawanda Sewer District, Mr. Hayes of New York State School of Orange County, and Mr. Hamm of Port Washington.

Of the reports submitted the one which achieved the highest rating and which incidentally closely approximated a perfect score, was that submitted by C. George Andersen, operator of the Rockville Centre plant. It is particularly significant that because of the complex nature of the plant, involving as it does both the activated sludge process and vacuum filtration process of sewage treatment, considerably more operating data were called for from Mr. Andersen than from any other operator in the field. This particularly is to his credit. That he was able to supply it, almost without fault or omission, came as a surprise to the committee. We did not anticipate that records at any plant would be quite so complete in the first year's operation of the Rating Plan. What Mr. Andersen has accomplished in this respect is within the easy reach of any operator in the state. He achieved it by applying persistently his individual energies to the purpose. I hope that his accomplishments may serve as an inspiration for other operators and that we may receive the permission of his Sewer Commission to publish his report as an incentive for other operators.

It now becomes my pleasant task to present to Mr. Andersen in behalf of our Association this award, which carries with it all of the esteem, respect, felicitation and congratulations which an organization of this kind can bestow upon a distinguished operator.

ACTIVATED SLUDGE CONTROL AT ROCKVILLE CENTRE AND THE PREVENTION OF BULKING *

By C. George Andersen

Supt. of Sewage Treatment, Rockville Centre, L. I.

Editor's Note.—Mr. Andersen was recently awarded the cup of the New York State Sewage Works Association for his Annual Report for 1935, on results of operation of the Rockville Centre Treatment Works. A report of the presentation ceremonies, with a forward by the *Chairman* of the Federation Mr. C. A. Emerson, Jr., is given elsewhere in this issue.

Mr. Andersen's present paper deals with operating technique used in the control of operation of the Rockeville Centre plant, and covers information not included in the annual report submitted to the Rating Committee. The Annual Report which won the prize is too voluminous to be reprinted here, but no doubt copies will be sent by Mr. Andersen to those requesting the report.

In operation of the Rockville Centre Sewage Treatment Plant (Fig. 1) we were continually troubled with bulking to various degrees until about two years ago. When difficulty was encountered, bulking would start each day about noon, decrease in the afternoon and start again at

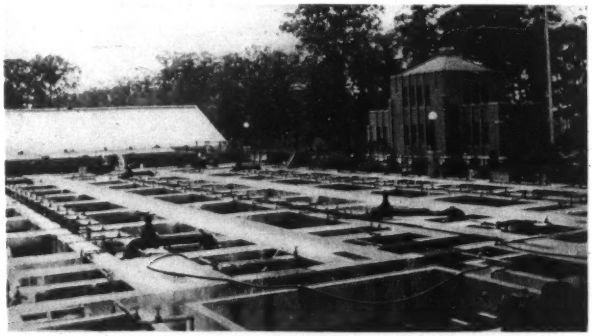

Fig. 1.—View taken from clarifiers showing aeration tanks, sludge tanks, covered sludge drying beds and blower house.

midnight until the bulking sludge had passed out of the tanks. At this time we were carrying 2,000 to 3,000 p.p.m. of solids in the mixed liquor in the aeration tanks and returning 20 per cent of sludge by volume. A great variety of opinions were expressed by visitors to the plant as to the cause of this bulking. Included in these reasons were—insufficient detention time in aeration and secondary settling tanks; septic raw sewage; low pH in raw sewage; the addition of supernatant liquor from the digestion tanks to the primary and aeration tanks; insufficient aeration due to lack of porosity in air tubes; unbalanced flow to clarifiers and the design of clarifiers and their equipment.

* Presented at the Spring Meeting of the New York State Sewage Works Association, Lido Beach, Long Island, June 11–13, 1936.

The possible effect of septic sewage was eliminated by prechlorination, which was provided at the pumping station as an odor prevention measure and has since been used to keep the sewage in a fresh condition. The pH was found to be normal. The sewage arrives at the plant with an average pH of 6.7 and after entering the first aeration tank unit it increases to 6.9 which is maintained through the aeration tanks. To determine the effect of a different pH, lime was fed to produce various pH readings up to 8.2 for a period of three months; this procedure did not show any beneficial results.

After the first year and a half the digestion tanks were well balanced and the supernatant liquor had no greater B.O.D. than a strong sewage. The inlet pipes in the final clarification tanks at one time were thought to have some effect on the bulking. The mixed liquor is introduced to the final tanks by four downdraft tubes in each tank. Each tube is located in the center of a quarter of the tank area. Deflectors were placed at the outlets of these inlet pipes to prevent the mixed liquor discharging against the walls, where the sludge rose when bulking occurred. These deflectors did not seem to be of any benefit.

After three years of operation the aeration tubes became clogged to the extent that back washing with water and even acid baths were of no benefit. We found that by putting a very fine crack in the sides of the tubes, operation could be continued, but with the production of considerable turbulence on the tank surface. The original tubes were in service for three and one-half years and new tubes of somewhat greater porosity have now been installed. Tests of samples of the new tubes (80 cu. ft. per tube) in parallel with the original tubes show that the new tubes, of greater porosity, have operated for a period of three years without apparent clogging.

Dissolved oxygen tests of the mixed liquor were not generally recommended but have been found to be a most practical test, and have proved to be a very important determination for successful operation. By this test we found that we had no oxygen in our mixed liquor, until the aeration was increased by cracking the aeration tubes, when dissolved oxygen was found to be present in small amounts. The question of balancing the flow through the two clarifiers was solved by dividing the aeration tanks into two separate units, one for each clarifier, with the flow divided to the two sections. Each section consists of three aeration tank units. The mixed liquor is weired to insure equal flows to the two clarifiers and the aeration tank design is very flexible, providing various flow combinations for any desired rate of flow.

Coagulation by chemicals was tried. Chemicals used included lime, aluminum sulphate, ferric chloride and ferrous sulphate. With each chemical a floc was formed and the sludge was made heavier, but in removing the sludge from the final tank the floc was easily broken up, requiring further application of chemical. Chlorine was used as a sludge rectifier in both return and mixed liquor. After checking reactions for about three weeks, we found that it acted like other coagu-

lants on the sludge, and at times we noted that it practically paralyzed the treatment, the effluent going off simply as aerated sewage.

After investigation of all the possible reasons for bulking that had been suggested or described, with their respective cures, and having heard that the same trouble had been experienced at other plants, the writer thought that he would start at scratch as no one seemed to have a satisfactory answer to the problem. The first problem was to find out what we were working with, what it was supposed to do, what was expected of it, and what was preventing the proper fulfillment of this job. I found that we were working with living matter existing under conditions every bit as exacting as those surrounding man or animal. In a sense it was a digestion process somewhat comparable to the human body and could not be expected to take care of iron, sawdust, concentrated chemicals, either acid or alkaline, or heavy fats. Like the human body it could not be over-fed, rushed in its feeding or expected to digest or break down septic material that would require more oxygen than could be provided, and keep in good condition.

What caused the septic conditions or the destruction of the aerobic or oxygen-dependent activated sludge? That there was death or septic action going on in this living matter was proof that we did not have sufficient oxygen, even when using our full capacity of blowers. The obvious answer was, therefore, either more tanks or a lessened demand for oxygen by reducing the number of organisms requiring oxygen, thus increasing the activity of the activated sludge. When the sludge is in good condition the settling is very positive and the settling test is one of the necessary tests in an activated sludge plant. Having surmised that too much sludge was being carried in the mixed liquor, the next problem was to determine how much sludge should be carried and how to control and regulate this quantity. While the settling test was useful, the determination of suspended solids was of little value as a control test, as it required too much time for completion I had at one time used a centrifuge in a laboratory and had also heard that it was used in sewage work and so proceeded to make a centrifuge out of a knuckle-jointed electric fan by bending the blades to hold the centrifuge tubes. This centrifuge is still in use. Samples were taken at various parts of the system and the control of the amount of sludge carried in the mixed liquor was worked out by installing a weir on the sludge drawn from the final tanks and regulating the rate of flow of return sludge in proportion to amount of solid as shown by the centrifuge. We worked out the concentration of sludge in the mixed liquor that would give the best results and found it to be between 800 and 1,000 p.p.m. This was determined by the dissolved oxygen test and by the determination of the amount of sludge that would give the best effluent without bulking.

Another complication with our activated sludge plant was the fact that all the sewage is pumped to the plant. This results in an intermittent flow, which produces a variation in the amount of sludge solids in the mixed liquor. This difficulty was overcome by regulating the

flow at the pumping station. This regulation is accomplished since the spring of 1932 by throttling the discharge valves on the pumps so that the rate of flow to the sewage treatment works is approximately constant throughout the day. We now have more uniform flow conditions through the aeration tanks than we would have with a gravity flow, unless some means of equalizing the flow were provided. This averaging of rate of flow also simplifies the regulation of the amount of solids in the aeration tanks. The desired amount of solids in the aeration tanks is maintained by checking the weir reading which shows the flow of returned sludge and frequent determinations by centrifuge of returned solids from the final tanks.

Table I is kept in the control room and is used by the operators for regulating the return sludge. Should the rate of sewage flow be 1.7 m.g.d., as shown by the plant Venturi meter, the operator finds from

TABLE I

Pump Rate M.G.D.	Weir Head in Inches	Return Sludge Volume in Gals.
1.3		
	2	74,000
1.4		
1.5		
	2¼	88,000
1.6		
1.7		
	2½	104,000
1.8		
1.9		
	2¾	120,000
2.0		
2.1		
	2¾	120,000
2.2		

Note: At the rate of 2.1–2.2 m.g.d., instead of increasing the flow over the weir, concentration of sludge is controlled by easing off on the excess sludge withdrawal, thereby returning more sludge per gallon of return. At this rate of flow the sludge is kept at about 14,000 p.p.m., as indicated by centrifuge reading of 8 to 9 ml. The reason this is done, is that a balance has to be maintained between the sludge gathering device and removal equipment.

Table I that the head on the weir which measures the rate of sludge return to the aeration tanks should be 2½ inches, indicating a rate of 104,000 gals. per day.

Sludge is pumped from the final clarifiers by centrifugal pump, with two suction pipes provided with one valve to each clarifier and two discharge pipes, one to the return sludge weir and one to the waste sludge outlet. The rate of return sludge flow over the weir (Fig. 2) and the density of sludge are regulated by the two pump discharge valves.

After the operator has set the rate of flow of sludge return by manipulating the pump discharge valve say with a 2½ in. head on the weir, he then checks the density of the return sludge. If this exceeds about 12,000 p.p.m., indicated by a centrifuge reading of 7 ml. in the 15 ml. tube (in the home-made centrifuge operating at 1,800 R.P.M. for five minutes), the waste sludge valve is partly opened. The operator makes a further check by running the solids in the mixed liquor. If this does not show about 800 to 1,000 p.p.m. dry solids, as indicated by a centrifuge reading of 0.3 to 0.4 ml. in the 15 ml. tube, further adjustments are made.

FIG. 2.—Weir used to measure sludge return.

At the present time sludge is returned by means of a centrifugal pump. The air lifts originally installed are kept for standby units, thus making duplicate pumps unnecessary. An interesting feature of the air lifts is the variation in the rate of flow with the density of sludge. Under the same conditions, with a constant rate of air flow, less sludge will be lifted when sludge density increases.

I have also been able to help plant operation by concentrating return sludge as much as possible. By this concentration and the use of the centrifuge method, used since the spring of 1932, the volume of return was reduced as much as 13 to 14 per cent. This reduction in volume has the effect of increasing the detention period in the aeration tanks. This is shown in the following table.

TABLE SHOWING INCREASE IN PERIOD OF AERATION EFFECTED BY CONCENTRATING RETURN SLUDGE

Rate of Sewage Flow		Detention Period in Aeration Tanks	
		Based on 20% Return by Volume without Regard to Concentration of Sludge	Based on Controlled Return of Concentrated Sludge (6% by Volume)
Min.	1.4 m.g.d.	6 hrs. 30 min.	7 hrs. 10 min.
Av.	1.7 m.g.d.	5 hrs. 15 min.	6 hrs. ——
Max.	2.0 m.g.d.	4 hrs. 10 min.	4 hrs. 45 min.

I have found that there is a very definite relation between the speed of the sludge removal apparatus and the rate of removal of sludge. When I was trying to use a 20 per cent return there was a positive short-circuiting of sludge in the clarifier. In other words, this rate was greater than the rate at which sludge could be moved to the outlet by the clarifier. Using a 20 per cent return, clear liquor was drawn down to the sludge outlet instead of sludge. By reducing the rate of returned sludge, concentration took place and a denser sludge was removed. Furthermore, the sludge did not build up on the sides of the tank where it could become septic and then bulk or be floated by the gas formed. It was also found that separation or settling took place in the columns of the air-lifts through which the sludge was drawn. This was very noticeable when the sewage temperature was under 50° F. and the solids in the clarifiers concentrated to 20,000 p.p.m. The solids in the return did not check with the known solids in the tank, showing that the sludge was not being completely removed. The clarifiers were then pumped out to see the exact picture of conditions. The result was very interesting. A large cheese-like formation was seen. The scraper and its rods were cutting through this formation and it was floating high enough over the sump in the tank to allow short-circuiting of the concentrated sludge. This sludge that was not being removed was becoming very septic. The sewage while going through the aeration tanks received treatment, but not the best because of the return of this septic sludge, and on entering the clarifiers the quality of effluent was further injured by the poisonous mass held in the clarifiers. After discovering what was happening in the clarifiers the removal of sludge by pump was started. The pump operation was slightly different from air-lift operation. The valves were regulated so that equal amounts of sludge were drawn from the two clarifiers. The pump discharge now flows over a weir and a connection is made on a discharge line so that excess sludge is drawn off continuously. The excess sludge drawn off is then decanted and concentrated to 50 per cent of its original volume.

In operating the aeration tanks and final settling tanks we have several control checks.

Settling Test.—A settling test of mixed liquor taken from the final aeration tank must give a result of less than 20 per cent by volume in 10 minutes. Our usual percentage is 12 per cent (Fig. 3). Should sludge continue to run over 20 per cent by volume in 10 minutes for as long as 24 hours, and should the sludge not be properly removed in the final tanks, indicating a real bulking sludge, dry lime is fed directly to the mixed liquor to the extent of 150 p.p.m. or 600 pounds to 500,000 gallons. This acts as a positive coagulant and the system can be purged of any septic or foreign matter in the sludge by removing a large percentage of it and very quickly giving the sludge a chance to revive itself instead of dragging it out for days, thereby having poor purification with possible bulking. After liming, the effluent will be somewhat inferior for about four hours, but shortly afterwards

will show very good purification. We have had no reason for using this extreme corrective measure for over a year.

Total Solids of Mixed Liquor and Return Sludge.—Total solid determinations are made daily on the return sludge and mixed liquor. The return sludge averages 12,000 p.p.m. and the mixed liquor 900 p.p.m. These concentrations are controlled, as described by checking

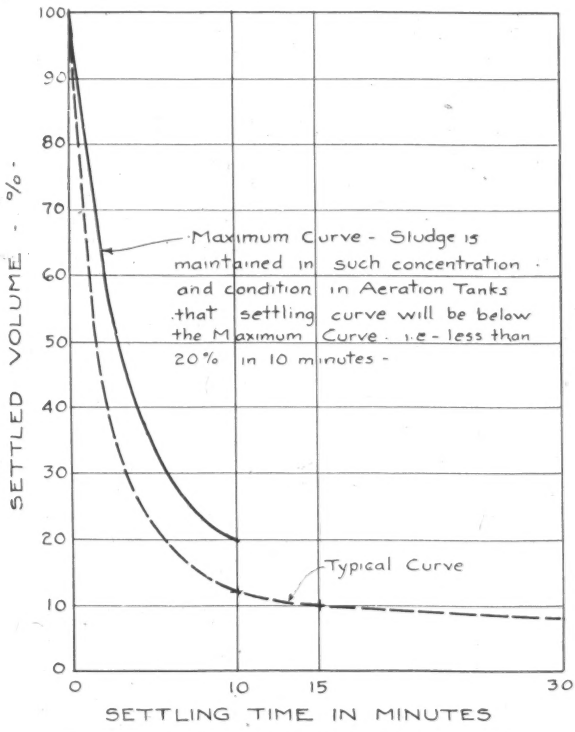

FIG. 3.—Typical sludge settling curve, Rockville Centre, N. Y.

the amount of solids by centrifuge and by weir readings on the return sludge. With this operation healthy sludge can be maintained having good settling characteristics. As an extra check at least once a shift a rod with sample bottles is put in the bottom of the final settling tanks and on the sides to check the sludge depth (Fig. 4). This precaution is taken on account of changes in personnel from time to time. The minimum amount of sludge is carried in the clarifiers which will insure constant areation of sludge in the system.

Dissolved Oxygen.—Dissolved oxygen determinations are made daily on the mixed liquor in the aeration tanks. These show dissolved oxygen ranging from a trace in the first unit to 3 p.p.m. in the last tank. We have found this test to be a positive and essential check on plant operation.

FIG. 4.—Sampling pole used to check final clarifiers.

B.O.D. and Total and Suspended Solids.—B.O.D. and solids determinations are made daily throughout the plant.

Back Washing Porous Tubes.—Porous tubes are back-washed at least once every two weeks winter and summer. New tubes have been installed and this washing procedure will be continued. Tubes are back-washed by connecting a hose to a T in the air pipe leading down to the aeration tube unit, which consists of two tubes.

Aeration Tanks.—Aeration tanks are checked frequently for the presence of any settled material at the bottom of the tanks. This is done by using a long iron hoe. No sedimentation in the aeration tanks has been observed since the agitation of the mixed liquor was increased by cracking the low-porosity and clogged porous tubes. After this cracking of the porous tubes the increased air pressure produced quite a turbulence at the surface and while there is considerable oxidation

going on under the surface I am a firm believer in the value of surface oxidation.

Primary Settling Tanks.—Tanks are operated with 1 to 2 feet of sludge kept in the hoppers at all times. By following this procedure concentrated sludge only is drawn from these tanks and pumped to the digesters. Holding 1 or 2 feet of sludge in the hoppers also results in making the sludge semi-septic before it is withdrawn. Sludge in this condition is most active in breaking down the excess activated sludge, which is still in an aerobic condition, so that when the two sludges are mixed the combined sludge fed to the digesters is in a condition that can be more quickly acted upon by the anaerobic sludge in the digestion tanks.

Operating Results for 1935.—A full-time chemist is employed at the plant and a large number of determinations are made throughout the day showing in detail the functioning of all the plant processes. The operating report for 1935 submitted to the State Department of Health required some 34 sheets so is too voluminous to include in this paper. In general the activated sludge portion of the plant is operating very near its design capacity of 2 million gallons per day. This portion of the plant is required to taken not only the total sewage flow but in addition the waste water from the vacuum filter plant. The air consumption has been reduced to about 2 cubic feet of air per gallon and at the same time the operating efficiency has been increased.

Odors.—The fact that the plant is being operated without nuisance is shown by the building of high class residences directly adjacent to the plant since its construction. The plant grounds are also used for a park and some of the percolation beds that are not required since the installation of the vacuum filters have been converted into a skating pond.

Conclusions

1. By operating the activated sludge plant as we do, sludge bulking has been eliminated by preventing conditions which we believe are responsible for bulking sludge.

2. Our treatment results are good. Observation of plant results will show that probably a little more aeration time would lower our suspended solids in the effluent from the secondary clarifiers.

3. A plant operating near its capacity needs close attention at all hours of the day and the personnel should be of the type that have an interest in their job.

DESIGN AND OPERATING FEATURES OF THE CANAJOHARIE AND HERKIMER, NEW YORK WASTES DISPOSAL PLANTS *

By Henry W. Taylor

Consulting Engineer, 11 Park Place, New York, N. Y.

A wastes disposal plant may be defined as one at which two municipal utilities are combined in one unit and two processes, namely, the treatment of sewage and the disposal of municipal refuse are carried on at the same plant to the mutual advantage of both processes.

For many communities one site for the disposal of these two municipal wastes has a distinct advantage. In many cases the combined construction of two units can be executed with considerable economy as compared with separate unit construction. In most communities a combination of the operating corps is desirable and in all municipalities the centralization of control is a marked asset. In the smaller communities where one operator would be required for each process in case of separate sites, a wastes disposal plant makes it possible to use one operator for both duties or one operator supplemented when required by occasional part time of another municipal employe who can be spared from either the street or water departments for a few hours.

There is another valuable benefit resulting from this combination, namely, that of the interchange of facilities. This combination of municipal utilities makes it possible to utilize the heat of the waste gases from the incinerator to maintain sludge tank temperatures for the digestion of the sewage sludge. The incinerator also provides final disposal for sewage screenings and air-dried sewage sludge.

In the case of the sewage disposal process alone, it is necessary to use the gas from sludge digestion for heating the sludge. With the combination plant, the incinerator performs this service and the sludge gas is released for another purpose. The most natural use for this sludge gas is to return it to the incinerator and use it as an auxiliary fuel or as an intermittent booster of incinerator temperatures to level off variations in the composition of the refuse. In larger plants the amount of heat required for sludge disposal represents only a small proportion of that available in the waste gases from the incinerator and in such plants there is little need for an auxiliary fuel. There thus remains a large heat value in incinerator waste gases plus the heat value of the sludge gas, both of which can be combined in boiler settings for the production of heat or power for revenue, or the sludge gas can be used for generating power direct for revenue or for sewage pumping. The interchange of heat values was introduced at the waste disposal plant at Canajoharie, which was built in 1934 under the auspices of the first P.W.A. legislation.

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Canajoharie Wastes Disposal Plant

The Village of Canajoharie has an actual population of about 2,600 people but because of a large industry, namely, the main factories of the Beech-Nut Packing Company, there is a considerable transient population from neighboring communities. The influence of this industry is evidenced not only in the sewage flow and sewage characteristics, but in the quantity and character of the municipal refuse. A complete sewer system and trunk sewer was built many years ago and discharged the raw sewage into the waters of the Mohawk River, now the Barge Canal. Municipal refuse had been collected by the municipality for many years and available dump areas had been largely exhausted. These dump areas were proving to be a nuisance due to the smoke, flame, rodents, and unsightly and insanitary conditions, while the necessity of obtaining additional dumping areas was becoming more urgent.

It was estimated from available data that the six-day average tonnage of municipal refuse would be approximately five tons per day and the nominal capacity of the incinerator was designed as 18 tons per 24 hours. A gauging of the sewage flow indicated about 250 gallons per capita and the sewage works was designed on the basis of a flow of 650,00 gallons per day, which agreed closely with the water consumption. The anticipated sewage flow has not materialized.

Description of Plant

In the main the plant consists of a concrete and brick building 50 ft. by 25 ft. in plan, with a northerly wing, 25 ft. by 15 ft, in plan, and a basement vault which enlarges the lower floor by about 8 ft. In the westerly part of the main building there is located the incinerator, the stoking and charging room floor together with the fuel storage, ash-can storage, toilet room, and the coal and gas auxiliary heaters. In the center of the main building are corridors, a shop and utility room, and the easterly portion of the building is occupied by an enclosed steel sludge digestion tank with Downes floating cover. The northerly wing includes the screen and suction well, the sewage pump well, the motor room, and the hot water storage tank room. The sewage settling tanks are contiguous to the building and extend to the north of the north wing. The sludge drying beds are located to the east of the settling tanks, and the effluent sewer extends northerly to the Mohawk River.

Basis of Design

The general details of design of the various features of the plant may be briefly stated as follows:

Sewage Works

Coarse Screens: 1½ in. spacing.

Sewage Pumps: of suspended type, each with capacity of 375 g.p.m.; total capacity of 3 pumps 1,600,000 g.p.d., automatically operated by float.

Primary Settling Tanks: with parallel sludge conveyor, designed for 2 hours detention for average flow of 650,000 g.p.d.; shallow type, level bottom, with two end hoppers. Tanks are equipped with multiple headers and end headers for effluent collection.

Sludge Ejector: capacity 100 cu. ft., hand operated.

Separate Sludge Digestion Tank: steel construction throughout, 1.25 cu. ft. per capita for 3,000 people, equipped with top jetter and jetter pump supplied with hot supernatant liquor; tank coils heated by incinerator. Collected gas returned to gas burners in incinerator or to gas-fired hot water boiler, or wasted to atmosphere.

Sludge Beds: open, 1 cu. ft. per capita for 3,000 people.

Incinerator

Incinerator: capacity 0.75 ton per hour, operated in test at 1 ton per hour; of multiple hearth type, equipped with coils in the combustion chamber for heating of water for hot water system for heating the building and the sludge.

Hot Water Storage Tank: 2,000 gallons capacity, connected to coils in combustion chamber of the incinerator, to the coils in the sludge tank, and to radiators in the building. Pressure operated under a closed system, with automatic thermostatic control of temperatures of water delivered to sludge tank coils.

Chimney: 70 ft. high from stoking room floor, lined throughout, and with flue 3 ft. in diameter.

Waste Oil Storage: 1,000 gallons capacity with oil filter.

The complete cost of the plant, including all charges, was about $58,000 and the grant will represent $15,800, leaving a municipal cost of about $42,000. For the design population of 3,000 the per capita cost for both processes was $14. A legitimate population rating, considering both the character of the sewage and the character and quantity of municipal refuse, would be nearer 4,000, which population figure would produce a net cost of about $10 per capita.

Operation reports are available since the 1st of February, 1934, and have been ledgered and summarized to May 1st, 1936. The following tabulation on page 796 gives the average quantities together with the absolute maximums and minimums of these quantities.

In this tabulation the power is used for the sewage pumps, sludge conveyor, compressor for ejector and the crane. The energy used for lights is increased by the flood lighting of the chimney from sunset to 12 o'clock midnight.

It may be added that the sludge tank room has been exposed to a minimum temperature of 40 degrees whereas an outside sludge tank would have been exposed to a surrounding temperature of minus 10 degrees for this same period.

The average temperature of the sludge digestion tank is a few degrees lower than will be the case in subsequent operation. This average temperature of 76 degrees results in part from the lack of an

TABULATION OF OPERATING DATA, WASTES DISPOSAL PLANT, CANAJOHARIE, NEW YORK
From February 1, 1935, to April 30, 1936—15-month Period

Description	Average	Max.	Min.
Power, K.W.H./Day	46.1	173.0	10.0
Lights	16.2	88.0	5.0
Temperatures, Outdoors, F°	54.4	110.0	−10.0
Sewage, F°	59.2	77.0	42.0
Sludge (Digestion Tank), F°	76.1	95.7	50.0
Incinerator, F°	1,250.0	1,900.0	961(Av.)
Sewage Flow, G.P.D.	333,000.0	1,250,000.0	40,000.0
Fresh Sludge, Cu. Ft./Day	135.8	350.0	0.0
Gas Production, Cu. Ft./Day	1,790.0	4,750.0	0.0
Digested Sludge Withdrawn, Cu. Ft./Day	22.63	—	—
Imhoff Cone, Influent, Milliliters	1.46	5.0	0.4
Effluent, Milliliters	0.13	0.5	0.0
pH, Sludge	7.15	7.8	6.8
Refuse Delivered, Total, Tons/Day	4.28	7.15	0.85
Village (Average for 6 days), T./D.	2.11	4.2	0.25
Beech-Nut (Average for 5 days), T./D.	2.22	4.0	0.5
Miscellaneous (Average for 6 days), T./D.	0.3	2.0	0.025
Residue from Incinerator, Cans	9.33	16.0	3.0
By Weight, Lbs.	933.0	1,600.0	300.0
Lime Added, Total Bags	81.0		
Lbs.	4,050.0		

adequate amount of sludge liquor when a very considerable depth of top sludge developed, and was indirectly affected by the high hot water temperatures resulting from additional readily-combustible material and the occasional need of discontinuing the sludge tank coil circulation. The average temperature for the furnace of the incinerator is a trifle low and this average has resulted from a former necessity of operating the incinerator so as to avoid the production of steam.

The quantities of fresh sludge per day have been measured volumetrically by the rise in the floating cover after fresh sludge has been discharged into the sludge tank. It will be noted that the per capita contribution of fresh sludge based on the actual population of 2,600 people is abnormally high. The figure for gas production for the entire period is not representative of an average condition inasmuch as gas production was not initiated until about April 1st, 1935, and the average figure for a 12-month period is 2,091 cu. ft. per day, giving a figure of 0.7 cu. ft. per capita per day for a population of 3,000 people which perhaps is a fair population equivalent. For the nominal population of 2,600 people, the per capita daily figure is 0.8 cu. ft.

The Imhoff cone tests of influent and effluent are taken only intermittently but the figures for the effluent are fairly representative of actual conditions.

With reference to the quantities of refuse, the total figures are correct on the basis of a 6-day week. The average figure for the Beech-Nut Packing Company's deliveries are on the basis of a 5-day week. For this reason the sum of the sub-totals exceeds the total.

Using the average figure of tons of refuse per day (4.28) and the nominal population of 2,600, the total pounds per capita per day of a 6-day week is 3.3 pounds. Using the village and miscellaneous quantities, the contribution to the refuse is 1.85 lb. per capita on the basis of a population of 2,600.

It will be noted that the incinerator residue is slightly in excess of 10 per cent. This residue is not weighed daily but the number of cans are recorded and the average weight for the contents of a can has been taken at 100 lb. as determined by test.

In the case of Canajoharie, a composite analysis of all the types of material delivered to the plant would be difficult to obtain and the only feasible method of analysis is that of taking the individual classifications for separate analysis and combining these analyses in the proportion in which they occur. An analysis has been made of a representative municipal delivery and it has been found that the physical analysis of this material may be stated as follows:

Glass, Tin and Metal	11%
Paper	30%
Garbage	59%
Total	100%

Of the above, an analysis of the garbage content indicates the following composition:

Moisture	54.0%
Combustibles	40.8%
Ash	5.2%
Total	100.0%

Discussion of Operation of the Plant

The Canajoharie Wastes Disposal Plant is operated by one man with occasional assistance from the street department for one or two hours at a time when the refuse load is excessive, when blowing sludge, cleaning tanks, stripping sludge beds, etc. Without intermittent assistance, which I estimate amounts to eight hours per week on the part of another man from the street department, one operator could not handle the work in an eight-hour shift. The residue from the incinerator is removed by the collection gang, with the assistance of the operator, after they have delivered their first load of refuse. The operator handles the cans of residue on the stoking room floor and the collection gang on the charging room floor receive these cans from the crane and dump them into the truck.

The engineer required that the board select the future operator of the plant during its construction, and the operator, Mr. Leland Benjamin, worked on the plant during construction for three months prior to its completion. The engineer retained supervision of the operation of the plant for three months after its completion and worked with the operator through a resident engineer, to make all necessary adjustments to meet operating conditions as found in practice.

There are certain items of experience in the operation of the Canajoharie plant which are of value in the design and operation of future plants and a few items of this experience will be mentioned, and, it will be noted, that the second plant discussed in this paper reaped the benefit of this experience.

The circulating piping between the headers of the coils in the incinerator combustion chamber and the hot water storage tank were designed as 1¼ in. piping with a No. 15 Thrush circulator to augment gravity circulation. The coils in the incinerator are exposed to waste gas temperatures, ranging from 1,000 to 2,000 degrees. Flash temperatures due to a highly combustible charge of refuse probably introduces momentary temperatures in excess of 2,000 deg. F. For normal refuse, the circulating system would have proved adequate. However, after the incinerator had been in operation for about two months the Beech-Nut Packing Company decided to discontinue baling paper and initiated the practice of delivering to the incinerator a large trailer of loose and fairly dry paper. The refuse from the Beech-Nut Packing Company also includes a large percentage of dry cartons, together with waste labels, advertising matter, and stale chewing gum collected from retailers. The response of temperature to this fuel is very rapid and the effect of a large carton of gum will extend over one-half to three-quarters of an hour. These industrial wastes also include quantities of wax paper which is highly combustible. With the delivery of paper formerly baled by the Beech-Nut Packing Company, the speed of circulation in the hot water coils was not rapid enough to prevent the formation of steam, and whereas in the original design there was a bit of skepticism as to whether the hot water storage tank would ever be heated from top to bottom to a temperature of 200 deg. F., it frequently became necessary to bleed hot water from the tank in order to cool it off and prevent throbbing in circulating piping and in the tank itself. This condition was eliminated by the installation of 2 in. circulating piping between the hot water storage tank and the incinerator coils, and a 2 in. Thrush circulator with a capacity of about 75 g.p.m. The cold water supply to the hot water storage tank was connected to the bottom of the storage tank. Further facility for the control of incinerator coil temperatures can be obtained by introducing the cold water supply into the return line of the incinerator coils so that when it is necessary to withdraw hot water from the system, cold water is introduced directly into the return main and reaches the incinerator coils at once.

The hot water supply to the coils of the sludge digestion tank is connected to the top and bottom of the hot water storage tank and these two connections are controlled by automatic diaphragm valves operated by air pressure and controlled by a thermostatic arrangement manufactured by the Tagliabue Company. A thermometer bulb is introduced into the supply main extending to the sludge tank hot water coils. If this supply exceeds a temperature of 135 deg. F., the hot water connection to the storage tank is closed and the cold water

connection is opened. These two diaphragm valves are in almost constant motion in an effort to maintain this control. When the temperature of the tank as a whole exceeds 135 deg. F., there is, of course, no cold water available to reduce the temperature of the hot water from the top of the tank. In new designs a mixing tank has been installed to reduce the frequency of temperature changes in the mixed water supply to the sludge tank coils.

It is apparent that no auxiliary fuel is needed at Canajoharie. Some of the gas is used in a gas-fired boiler in winter to maintain temperatures in the hot water storage tank when the incinerator is not in operation. During the summer months, most of the gas is now wasted for lack of any practical use.

Two gas burners were installed in the incinerator for the use of sludge gas to maintain temperatures depreciated by a normal character of municipal refuse, to warm the furnace prior to installing the first charge, and in general, as an auxiliary fuel for the incinerator. The available gas pressure, by weighting the Downes cover, could be increased to about 12 inches of water. Sufficient use of the gas burners has been made for purely experimental purposes to determine the best type of burner for this purpose and the pressures under which it should be operated. It has been concluded that a special burner operating under about 4 lb. pressure is necessary for this service. It also became apparent on a theoretical basis that any auxiliary fuel supplied to an incinerator is demanded only periodically but when demanded is required in considerable volume, and the idea of gas storage for this service immediately suggested itself. In the second plant discussed in this paper gas compressors and gas storage have been provided so that gas, when used, could be delivered to the incinerator at a rate of more than ten times the rate of its production.

The collection of waste oils from garages has kept these oils out of the sewage but the feasibility of the use of such waste oil in an incinerator without an oil burner is questionable. The waste oil tank was installed on an experimental basis to determine the amount of these oils which would be collected in a month. In the case of Canajoharie, no auxiliary fuel is needed. Waste oil is used in secondary road improvement.

Just prior to the design of the Canajoharie Wastes Disposal Plant, a considerable amount of work had been done in connection with the collection of effluent from the secondary settling tank of activated sludge plants. We are all familiar with the apparent recoil of small particles of suspended matter against the end wall of a settling tank provided with an end weir. The idea of collecting the effluent at a point back from the end wall of the settling tank had already been fairly well established. In the Canajoharie plant, the weirs were eliminated, and there were substituted therefor effluent headers, located so as to draw a regulated percentage of the effluent from headers located at about the third point, the center, and at a point about 5 ft. away from the end wall of the settling tanks. In addition to the multiple

effluent headers, there were supplied submerged end headers. These various headers were controlled by valves and separate weirs so that any one or all of them could be used at the same time. No laboratory tests have as yet been made to verify the relative efficiency of this multiple header scheme. It is thought, however, that this plan is better suited to a deeper tank than the Canajoharie tanks, and to a sewage containing a larger percentage of heavy suspended matter. The Beech-Nut Packing Company materially influences the character of the municipal sewage, but principally by matters in solution or semi-solution, which are not amenable to preliminary sedimentation. The tanks are frequently blackened by a solution which periodically reaches the plant in the afternoon, and, which, I think, is connected with the manufacture of chewing gum. It will be noted, however, that the settlable solids and sludge do not seem to be seriously affected by these industrial wastes.

The digested sludge withdrawn from the sludge digestion tank has proved to be consistently of prime quality. A considerable volume of floating sludge has frequently accumulated and has reached a depth as much as five or six feet. At such times the available sludge liquor is limited, and on one occasion became so thick as to clog the small monobloc jetter pump. Due to a considerable and unnecessary delay in obtaining a hose fitting, when the plant was first put into operation, the jetter could not be used and consequently floating sludge accumulated when gas action was first initiated, causing a few weeks of sludge tank trouble. The sludge tank has again demonstrated the need for an active agitation of floating sludge and has shown the possibility of dropping this floating sludge by a jet of hot sludge liquor with sufficient pressure to thoroughly break up this sludge mass, release the gas, and make settlement possible.

Incinerator operation has suggested the need for protection of stoking doors to save refractories. A new operator will chip many corners of refractories until he has become accustomed to his new work and the refractories at door openings, with this type of multiple hearth incinerator, should be amply protected from the wreckage of heavy stoking, or the openings should be splayed at a considerable angle.

The operation of the Canajoharie incinerator has again proved the necessity for some quick charging device. The writer has frequently watched the waste gases escaping from the Canajoharie chimney, and, except for periods of charging, these gases give the appearance of heat waves without appreciable color. However, if the charging chutes are left open for any considerable number of minutes, the character of these waste gases immediately takes on a perceptible color. A charging chute can not be left open while the operator slowly drags refuse forward to it, and the incinerator should receive a stored charge quickly.

The question of fly ash has been studied, although this factor is of no importance as far as nuisance is concerned at Canajoharie. A rigid draft control must be maintained by the operator and this con-

trol must be accommodated to the temperature and the humidity of the air and the material being burned. It is apparent that with a suitable combustion chamber and draft regulation, there is no necessity for fly ash. Draft regulation should not be applied by means of the main flue damper, which is necessarily cumbersome, but should be supplied by air admission valves. At Canajoharie no need for forced draft has been experienced, although with the average municipal refuse, it is considered that forced draft should be available, although, perhaps, infrequently used. In a plant of this size, pre-heating of air supply is a questionable necessity, but the increase in the volume of incoming air by heat expansion is obviously a great asset from the mechanical angle of bringing excess air in quick contact with each particle of the refuse.

Sewage screenings which have been raked from the screen onto a concrete floor, without ample opportunity for slow drainage, when delivered to the incinerator, involve a considerable amount of unnecessary stoking. An attempt was made to drain these screenings by placing them in a can with a porous bottom, all of which proved unsuccessful. Later, the concrete floor adjacent to the bar screen was chipped out and a perforated steel plate was set above the concrete so as to permit of drainage below it. By this method the screenings were dewatered sufficiently to avoid trouble in their incineration in conjunction with the ordinary run of municipal refuse.

Air dried digested sludge cake has been burned in the incinerator in conjunction with the municipal refuse and this addition of sludge cake has introduced no complication in the operation of the incinerator. In the case of Canajoharie, the sludge cake will be utilized for some time on the sewage works site and any surplus will be removed for use on parks, golf courses, lawns, etc. In other words, combustion of the sludge cake is not necessary or desirable.

The sludge digestion tank was designed on the basis of 1.25 cu. ft. per capita for a population of 3,000 people. This conservative figure was adopted in view of no exposure to outside temperatures and, also, because of the expense of housing a tank of larger dimensions. State Departments of Health have, as a rule, questioned the practicability of this low per capita capacity, and as a matter of ease in operation, it is the writer's opinion that, in the case of a single tank, larger capacities should be used for convenience in operation. For a single tank, a mimimum of 1.75 to 2 cu. ft. per capita should be allowed.

The Canajoharie settling tanks illustrate the effect of intermittent pumping. The preliminary estimates of sewage flows were supplied to the writer and proved to be excessive. It was the intention of the design to supply a pump unit which would meet the minimum flows and consequently one pump would be in continuous operation. In actual operation, one pumping unit is not maintained in constant operation, and the flow to the settling tanks is intermittent. The discharge valves of the operating pumps were throttled down so as to produce continuous operation of one pump, with a beneficial effect upon the efficiency

of the settling tanks. However, this adaptation increased the power consumption considerably and the plan is not uniformly carried out. This effect of intermittent pumping to a settling tank is especially undesirable for a level bottom, shallow, settling tank, and it would have far less effect upon the efficiency of the tank if the tank were deeper and had a sloping bottom with the maximum depth at the outlet end. The effect of such design has been demonstrated in two other plants, especially at St. Albans, Vermont, where operation at 200 per cent rated capacity has produced very little change in the resulting effluent.

In the case of the Canajoharie incinerator, the air supply is admitted through the ash door. In the winter time when the outside temperatures are low, the tendency of the operator is to close the window in the stoking room and draw the necessary air from other parts of the building. If the stoking room door is closed, the air supply must come through the ash chute from the charging room above and this produced a rather disagreeable draft from above, striking the operator at the shoulders. The operator saw to it that a metal cover was made for the ash chute and subsequently the air supply was drawn from other rooms in the plant building. There resulted several noisy doors and a negative draft in the sludge tank room which affected the operation of the turbine ventilators in the sludge tank room roof.

The above serves to further illustrate the need for air intake arrangements even in the small plant, whereby the draft can not only be intelligently regulated, but by which it can be conducted directly to the ash pit in closed conduits. The above experience resulted in providing air ducts in the Herkimer plant which can either take their suction from the stoking room floor near the ceiling, or from the charging room through a closed duct to the ash pit.

Herkimer, New York

The following discussion will describe the methods of future operation for a wastes disposal plant at Herkimer, N. Y., which has been developed upon the experience gained by the design and operation of the Canajoharie plant.

The Herkimer Wastes Disposal Plant is being installed under a loan and grant from the P.W.A. and work was started on its construction on May 1st, 1936. The general contract for the work involved about $70,000 and the project will be completed for a total cost of approximately $78,000 including all charges. The grant in this case is $32,000 leaving a municipal cost of $46,000 and, with a population of 10,000, making the per capita cost for the combined plant $4.60. On the basis of the future population of 12,000, for which the plant has been designed, the per capita cost for both processes is less than $4.

A complete sewer system was installed in Herkimer about 40 years ago, and a 21 in. trunk sewer installed discharging into a circular well about 17 ft. in diameter from which an outfall extends into the Mohawk River. The sewage works site is located on property now owned by the

village, which had been built up between two rubble retaining walls to escape the flood water levels of the Mohawk River. This area, between these two retaining walls and an embankment between a highway bridge and a flood bridge, is being developed for the new wastes disposal plant. The sewage from the existing trunk sewer will be lifted about 17 ft. to the preliminary settling tanks and the purified effluent will be discharged at an elevation beyond the range of any recorded flood stage of the river. Municipal refuse is now collected by contract and delivered to a dump area. The refuse now collected may be classified as containing an undue percentage of garbage. New regulations and ordinances will develop a better proportion of refuse and readily-combustible materials. Available records of weights collected indicate a daily average of less than 7 tons or 1.4 lb. per capita. This weight will be increased to better than 1.5 lb.

Basis of Design

In brief, the two processes include the following main features:

Sewage Works

Bar Screens: manually raked, 1¼ in. clear spacing. Ventilated draining plate for screenings. Hoist for delivering drained screenings to the incinerator chute.

Pumping Equipment: 3 suspended type, vertical shaft, sewage pumps, float operated, capacity 450 g.p.m.; total capacity 2,000,000 g.p.d.: pump discharges arranged to provide aeration to release sulphide gases.

Primary Settling Tanks: with parallel sludge collectors, for 2 hours detention for 1,200,000 gallons per day; sloping bottom, 2 sludge hoppers each, double weired effluent trough away from end wall, and effluent header at low elevation at outlet end of tank; separate weirs for effluent troughs and effluent headers; main weir with recorder.

Sludge Ejector: in control house between sludge tanks, 180 cu. ft. capacity, discharging to distributing manhole on roof.

Effluent Sewer: effluent again aerated by means of ventilated weir chamber with 4 ft. drop, and returned to existing effluent sewer.

Separate Sludge Digestion: 2 tanks; 1 with fixed cover, the other with Downes floating cover, with 5 ft. skirt to act as gas holder. Tanks arranged to work in parallel or in series and in either direction. The two tanks supply 1.75 cu. ft. capacity per capita, and gas storage of 4,000 cu. ft.

Sludge Drying Beds: uncovered, 1 cu. ft. per capita, for 10,000 population; beds at an isolated location; arranged for expansion.

Incinerator

Incinerator: Decarie Type with basket grate, 1 ton per hour capacity, 27 sq. ft. of burning grate; large ell-shaped combustion chamber, forced draft for 40 cu. ft. per second of air at outdoor temperature,

flat suspended arch construction; refractory type with steel case; air intake, from top of refractory roof, providing a certain degree of pre-heat, or from charging room; basket grate connected to headers in the circulating system of the hot water system.

Hot Water Storage Tank: 3,000 gallons capacity, with circulation through the headers of the basket grate in the incinerator, supplying heat to the building and to the sludge tank heating coils through thermally controlled diaphragm valves to give it constant temperatures from 115 to 135 degrees.

Chimney: 75 ft. high, shaft lined throughout, 3 ft. in diameter.

Sludge Gas Burners: 2 sludge gas burners just above burning grate; 1 burner near top of basket grate, 1 in combustion chamber; each burner with a capacity of 800 cu. ft. per hour under 4 lb. pressure; burners especially designed for incinerator service; two gas compressors taking their suction from the Downes cover, delivering gas at 4 lb. pressure and each with a capacity of 1,800 cu. ft. per hour.

Description of Plant

The plant building is being built of concrete and brick and is located about 100 ft. from a main artery of the municipality, with a road plaza in front of it on a level with the tributary thoroughfare. The building is 50 ft. by 25 ft. in plan. The stoking room and the charging room are located in the western part of the building and a vault on the south side of the building includes a basement toilet with shower, a fuel storage, ash can storage, shop, etc. Adjacent to the incinerator is located an auxiliary coal heater and a gas-fired hot water boiler, both of which are piped in series with the basket grate headers. The forced draft fan is located just east of the incinerator and this fan can draw either through registers in front of the top of the incinerator casing or can draw through a duct from the charging room above. In the center of the building are the basement and ground floor corridors and in the basement are located the motor room and the screen room. The dry pump well is just below the motor room and the hot water storage tank room is above the screen room. On the ground floor the charging room for the incinerator occupies the westerly half of the building and the central corridor, office, and tank rooms occupy the remaining area. The settling tanks, of which there are two, are built adjacent to the east elevation of the plant building proper, while the two sludge tanks are located east of the settling tanks, and their location allows for the installation at a future date of a third settling tank unit. Interconnecting piping between the plant building, settling tanks, and sludge tanks is carried to the control house, which is constructed between the two circular sludge tanks.

The area in the vicinity of the plant building proper and the settling tank will be fenced in and a circular fence extended around the open sludge tank, which is equipped with a Downes floating cover.

Proposed Operation

The incinerator charging chute is provided with a heavy cover of cast iron and concrete which will be lifted from the chute opening by the crane. Below this cover the chute is equipped with two leaf balanced doors which automatically open with a load of about 10 lb., dump the load, and swing shut again. The basket grate will have a charging capacity of about 2 cu. yd. so that the bulk of the first load delivered to the plant can be placed directly in the basket grate. To the right of the charging chute on the charging room floor is, for lack of a better name, a kindling chute, into which will be thrown readily-combustible material for use in starting fires or augmenting temperatures on the burning grate.

This charging chute arrangement avoids the depreciation of furnace temperatures resulting from slow charging by the operator and the exposure of the furnace to cold air while the operator may be pulling material forward toward the chute, with the main chute cover off. The balanced doors will exclude the bulk of the inrushing air at any time the main chute cover is lifted. The basket grate affords an opportunity for charging in bulk, so that many trips by the operator from the charging floor to the stoking floor will be eliminated. The kindling chute avoids the necessity of attempting to feed readily combustible material through the basket grate to the burning grate below, the discharge opening of the kindling chute being only a few feet from the burning grate stoking door.

In the corner of the charging room there is located a 2 ft. by 2 ft. shaft equipped with register and this shaft is connected with the suction of the forced draft fan. When so desired, the operator can open this register and close an air controlling valve in the top of the incinerator and take air from the charging room. This air can either be forced draft with the use of the fan, or it can pass through the fan with natural draft. The charging room floor and the north passage is equipped with a monorail electric crane, which will be used to lift cans of residue in the stoking room floor level, the screenings from the screen room level, the heavy chute cover, etc.

The air inlet in the incinerator is at the back of the ash pit and all air will pass through the fan housing whether the fan is in operation or not. The fan will have a capacity of 1,800 cu. ft. per minute, providing better than five pounds of air per pound of refuse. A vent for the stoking room is also provided by a port in the top of the flue casing, which will exhaust air over the suspended roof of the flue, to the air space between the brick lining and face brick of the chimney.

In starting the incinerator, dry combustibles will be placed on the burning grate and about 1½ cu. yd. of refuse will be discharged into the basket grate. During the initial stages of operation the forced draft will be used and the gas burners will be utilized to boost initial starting temperatures, while the furnace is coming to normal operation. Three of the gas burners are in the grate area of the incinerator and will pro-

ject flame in the direction of the draft. One booster burner will be installed in the first compartment of the combustion chamber and this burner is controlled in front of the furnace and will be used intermittently throughout the incinerator operation to insure complete combustion of the escaping waste gases. It is probable that gas production will be at the rate of 300 cu. ft. per hour, whereas, if needed, the gas may be used in the incinerator, at a rate of 3,200 cu. ft. per hour, by drawing from storage in the floating cover and, if necessary, from the upper 1½ ft. of the fixed cover sludge tank. It is proposed to use the gas burners for starting the furnace, for perhaps the first half hour of operation and to use them intermittently thereafter in case of any temperature drop. With all four burners in operation, it will be possible to contribute to the furnace, from an outside source, 2,200,000 B.T.U. per hour. In winter a small amout of gas will be used in the gas-fired hot water boiler to maintain temperatures in the hot water storage tank when the incinerator is not in operation. The second auxiliary, namely, the coal-fired boiler, will probably never be used after gas production has been started in the sludge digestion tanks, but is installed as a standby unit for extremely cold weather to prevent the chilling of the basket grate and assist in maintaining hot water storage during the night and prevent exhaustion of heat storage by building radiators.

The flue damper is designed to be either open or closed and is operated by a worm gear wrench mounted on the face of the incinerator case. The control of air supply to the incinerator is provided by an air admission valve in the discharge of the forced draft fan, through which all air is supplied to the furnace, whether the fan is operating or not.

Two gas compressors located in a room adjacent to the furnace room take their suction from the main gas line extending to the control house between the sludge tanks and discharge the gas at 4 lb. pressure into a main which feeds the four gas burners. Each of these gas compressors is equipped with a by-pass between the discharge and suction lines, so that in case their full capacity is not being used by the burners, the excess gas will be returned to the suction line. A gas flame trap is provided on the gas line to the gas burner after the two discharges have been combined.

The hot water storage tank has a capacity of 3,000 gallons. The cold water supply does not enter the tank directly but is connected through a pressure reducing valve and pressure relief valve to the return system between the hot water tank and the headers of the basket grate. These headers of the basket grate are provided with a drip which makes it possible to bleed hot water, if necessary, and this hot water is replaced by cold water from the municipal water system, direct to the return header of the basket grate. Each hot water header of the basket grate is supplied with an air eliminator which will automatically, by float operation, release a needle valve and permit the escape of air or water vapor. The return line of the incinerator is

equipped with a 2 in. Thrush circulator with a capacity of 75 gallons per minute and both supply and return piping are 2 in. in accordance with the experience at Canajoharie.

The supply line for the building heating system is taken from the top of the tank to supply water at about 200 deg. F. The heating system supplies radiators in the office room and the motor room. Toilet rooms, lavatories, etc., are provided with hot water from the return lines of the heating system. The sludge tank coils are supplied from a mixing tank adjacent to the hot water storage tank, and this mixing tank receives hot water from the top of the tank and cooler water or return water from the coils for the cold component. If the outgoing water from the mixing tank is too warm, the Tagliabue thermal control will close the hot water inlet until this temperature has subsided to the desired level. As previously stated, when the hot water storage tank becomes heated throughout, the return water from the coils will supply the cool water for the mixing tank, and hot water will be introduced only intermittently to maintain the desired temperature Hot water circulation for the sludge tank coils will be provided by a small Westco pump with a capacity of 20 gallons per minute and provided with a by-pass so that the capacity of this pump can be reduced at will. The hot water circulation piping between the tank room and the control house will be covered throughout. In the control house, valves are available for throwing all of the circulating water to the coils in one tank or such sub-division of the heat as may be desired. No meter has been provided for this circulating water for the sludge tank coils, inasmuch as a great deal of trouble has been experienced in several plants with such meters which, apparently, are out of order a considerable percentage of the time. It will, therefore, not be possible to compute accurately the B.T.U.'s admitted to the sludge tank. The Tagliabue thermal control appartus is operated by air pressure, and, adjacent to the storage tank and mixing tank, a small, feather valve, air compressor will be supplied with storage tank for 150 lb. working pressure, which will furnish air, reduced to 15 lb. pressure, for the operation of the diaphragm valves.

Water for flushing and sprinkling is available from two outside hydrants, one being located east of the settling tanks and the other west of the plant building.

The separate sludge digestion tanks are arranged so that they can be operated independently, in parallel, or in series. The present plan of operation is to utilize the fixed cover tank as a primary tank. Unless the required gas storage is in excess of the capacity of the deep skirt on the Downes floating cover, which capacity is 3,000 cu. ft., no gas will be stored in the fixed cover tank and the overflow level of these tanks will be arranged to that the top sludge will be in contact with the underside of the fixed roof when the gas system is under the required pressure to lift the Downes cover free of the sludge. However, each sludge tank has been provided with two overflows, the lower one of which is valved. In case it is desired to store gas in the fixed

cover tank, the valves for the lower overflows will be closed, thereby throwing the upper level overflows into service. The gas line from the fixed cover tank is equipped with a pressure reducing valve so that the gas will be held back in this tank and can not escape to the floating cover tank except as a certain head is established, which is approximately three feet of water. As this gas is collecting in the fixed cover tank, the two tanks will operate together as a U tube by means of an interconnecting liquor pipe. As gas accumulates under the fixed cover and cannot escape through the pressure reducing valve, sludge liquor will be displaced to the other sludge tank until a hydraulic head of say three feet has been established, at which pressure the gas will then escape to the floating cover tank and supplement the storage under this cover. A bypass around the PRV permits escape of gas to the floating cover or to compressors. Under the expected method of operation, there will be continuous submergence of floating sludge in the primary or fixed cover tank. The dome of this latter tank is provided with a jetting nozzle which may be rotated in any direction and can be raised or lowered to reach different strata of sludge. This jetter will be supplied with 100 g.p.m. of hot sludge liquor, under about 50 lb. pressure, from one or the other of the tanks.

Fresh sludge is manually drawn from the two sludge pockets in each settling tank into a slot in the central wall of the tank and is delivered by gravity to a steel ejector tank, 6 ft. × 6 ft., in the control house, between the two sludge tanks. The air to this ejector tank is manually controlled and displaces the sludge through a discharge main to a manhole on the control house roof, from which it gravitates to one tank or the other as may be desired. The discharge line from the ejector has no check line and the contents of the force main is returned to the ejector. This backflow keeps the bottom of the tank clean. The force main discharges into the atmosphere and in case the operator applies too large a quantity of air, this excess will escape to the atmosphere and not be injected into the sludge tanks. Air for the operation of the sludge ejector is taken from a large pressure tank which receives the discharge of two, feather valve, monobloc air compressors, each with a capacity of 7.5 cu. ft. of free air per minute.

Two types of diaphragm vacuum relief valves are supplied on the gas system. Should the operator run the gas compressors when no stored gas is available, air will be admitted near the compressors.

The sewage screenings will be raked from the coarse screens, drained on a perforated steel plate onto which they have been raked, and the box cover of the drainage plate and screen closed. The following day these drained screenings will be lifted by a crane and discharged on to the basket grate of the incinerator. Digested sludge is discharged by gravity through 600 ft. of piping to isolated air drying beds. The air dried sludge will be used at the site, for park lawns and by individuals for lawns and golf courses. The dried sludge can be burned in the incinerator if desired. Ashes and residue from the incinerator will be delivered to the existing dump.

Discussion of Taylor Paper

By C. C. Agar

Senior Sanitary Engineer, New York State Department of Health, Albany

In designing a waste disposal plant combining the treatment of sewage and the disposal of municipal refuse, Mr. Taylor has developed a logical idea which has possibilities of economical application in many communities. There are several obvious advantages of a combination plant which have been pointed out.

One feature of such a plant which appears important to me is its popular appeal to the taxpayers of a community. There is a certain fascination in the idea of utilizing one process to aid the other by interchange of facilities. And the possible economies in construction and operation provide a convincing talking point.

In considerinig sanitary improvements for any community it is usually not difficult to secure improvements for a water supply system. The water supply is a utility in which each taxpayer has a personal interest and concern and furthermore is usually a self-supporting or revenue producing utility. An incinerator project may be somewhat more difficult to put across, but when unsightliness, odors, rodents, smoke and fires from the local dump, are considered there is good argument for its construction. Sewage treatment on the other hand is usually one of the more difficult projects to "sell" to a community. It does not produce revenue and the discharge of untreated sewage into streams usually does not materially affect the health or welfare of the community itself. When, however, a sewage treatment project can be joined with a refuse and garbage incinerator and their construction and operation can be combined, effecting material economy, such an improvement, because of its popular appeal, is more easily promoted.

Some of the features of the Canajoharie plant are of unusual interest. Although a few of the anticipated advantages have not been realized, on the whole the plant has operated efficiently and effected satisfactory disposal and treatment of the sewage and refuse from the village, and the results have more than justified its installation.

The use of sewage gas as auxiliary fuel for the incinerator at the Canajoharie plant has not been necessary because of the high proportion of combustible material available. Experimental use of the gas burner, however, showed that it was difficult to maintain the flame with the burning equipment provided. During the charging operation the flame was frequently extinguished. This difficulty can be accounted for, in part, when the characteristics of sewage gas are considered. A high proportion of methane and considerable carbon dioxide produce a flame which has a low rate of propagation as compared with the usual municipal gas supply. This means that special burners, which can be regulated carefully for air and gas mixtures, and a supply of gas under pressure has promise of satisfactory performance, and you will note

that those features are included in the design for the Herkimer installation.

The use of compressed air for operating a sludge ejector has been utilized before but is not a common type of installation. It should be noted that provision is made to prevent the discharge of air into the digestion tank by an air relief on the sludge line. This is particularly important because of the explosion hazard which might be created if air was admitted to the digestion tank.

Although the digestion tank is located in a separate room, the tank is equipped with a floating cover and there is, therefore, some loss of gas from the tank into the enclosure. A ventilator was provided in the room but there have been times during the winter, when the windows of the building were closed, that the draft of the incinerator stack was sufficient to reverse the ventilator draft. It is not considered safe or satisfactory to house a floating cover digestor unless positive and safe means of ventilation are provided. As the purpose of housing is to conserve heat and to protect the tank from the elements it can be appreciated that an effective ventilating system would reduce these advantages somewhat.

With regard to the advantages of multiple effluent headers and the modified weir take-offs at the various points in the settling tanks, no complete tests have been made. However, the settlable solid content of the effluent has been consistently low. It appears logical that these tanks would produce a better effluent than a single effluent weir at the end of the tank. Whether such effluent would be as good as could be obtained with trough weirs at the same points in the tank length is questionable. With either arrangement it is necessary to provide hand skimming of the settling tanks as the flight conveyors can not traverse the surface because of interference.

In the operation of the sludge digestor at Canajoharie, difficulty was experienced from an excessive scum formation of gas-lifted sludge, as pointed out by Mr. Taylor. This is a condition which frequently develops in digestors during the first year or so of operation. It appears to be a characteristic of the breaking-in period, and there are few plants where some such difficulty is not experienced. The necessity of careful control and provision for stirring or agitation during this period is evident. After obtaining the necessary fittings so that the jetter extending through the gas dome could be utilized, the scum problem was soon corrected.

Experience in the operation of the Canajoharie plant has resulted in a number of improvements and changes in the design of the Herkimer plant. While it will be some time before the Herkimer plant is placed in complete operation, it is expected that the difficulties encountered at Canajoharie, all of a more or less minor or temporary nature, will be avoided. Results of operation of the Herkimer plant will, therefore, be awaited with interest.

OPERATION OF THE DEARBORN, MICHIGAN, SEWAGE TREATMENT WORKS *

By Edmund C. Miller

Formerly Chief Engineer of the Division of Sewage Treatment, Dearborn. Now Engineer with George B. Gascoigne, Consulting Sanitary Engineer, Cleveland, Ohio

The sewerage system in Dearborn is divided into two general sections, the East Side and the West Side. The East Side system takes sewage from the city of Detroit, the eastern part of Dearborn, and from the Ford Motor Company. The East Side plant consists essentially of a pumping station, grit chambers and Imhoff tanks, which have been converted so that they are used as primary sedimentation tanks. Sludge is withdrawn from these tanks at more or less regular intervals and pumped to the West Side plant for final disposal. The western section of the sewerage system receives sewage from the Wayne County Home and the western part of Dearborn. The West Side plant consists essentially of a pumping station, flocculators and Dorr clarifiers, equipped with magnetite filters. For a more complete description of these plants reference is made to other articles that have appeared before, by Owen,[1,2] Miller,[3] and Buswell.[4]

PWA Additions and Alterations

In 1934 certain changes were made to both the plants under a PWA contract. The principal change made at the East Side was the installation of mechanically cleaned screens with the bars having ¾ in. clear openings. The screenings are carried to the top of the racks and dropped onto a belt conveyor, which feeds them to a Jeffrey grinder. From the grinder the material drops back into the raw sewage, and from then on is handled as sludge. The contract cost of the East Side changes was approximately $18,000.

At the West Side plant more adequate provision was made for chemical application and flocculation of the sewage. The chemical feeders are Omega precision type and the lime feeders are Omega bucket elevator type. Chemical solutions used are such that each gallon of ferric chloride solution contains one pound of $FeCl_3$, and, similarly, the lime solution contains one pound of CaO per gallon. The chemicals in this form are discharged into the sewage, lime first, followed by ferric chloride, in a mixing chamber having a detention period of about 6 minutes. Agitation is accomplished by means of 140 cu. ft. per min. of air introduced into the sewage through Aloxite diffuser tubes, located along one side of the mixing chamber. The sewage passes around a baffle into a Dorr flocculator, where it is mixed at a velocity of 1.8 ft. per second for a period of 20 minutes. From the flocculator the sewage is carried through an inverted syphon and

* Presented before the Tenth Annual Conference of the Pennsylvania Sewage Works Association, State College, June 24, 1936.

a Dorr Sifeed into the center of the clarifier. All of this equipment is installed in duplicate. Theoretically the detention period in the clarifier, at the designed rate of flow of 4,000,000 gallons per day, is somewhat over one hour. However, dye tests have indicated the detention period to be actually less than one-half hour.

The sludge mixing equipment originally installed in the plant was quite crude, consisting merely of sumps 6 ft. in diameter by 17 ft. deep, into which a batch of 3,000 gallons of sludge was drawn and the chemicals added. The sludge was then agitated for a short period of time by means of compressed air, after which it was pumped to the pans of vacuum filters. The changes made consisted of the installation of a bucket elevator in each of the two sumps. The elevators discharge to two rotary drum mixers, similar in type to concrete mixers, and which have a detention period of about 10 minutes at normal rates of operation. These mixers were on hand, otherwise a different type might have been used. Each bucket elevator drive has an associated, mechanically interlocked, ferric chloride feeder and a lime feeder. By this means the chemical dosing is proportioned to the amount of sludge that is being handled by the bucket elevator. Superimposed on this control is a manual control for setting the rate of dose required. Under this same item of the PWA contract, two rubber lined ferric chloride storage tanks, each having a capacity of 12,000 gallons, were installed. Installation of these tanks enabled a considerable saving in the price paid for ferric chloride in that it can be delivered in carload lots as compared with carboys previously used. All ferric chloride is diluted when placed in storage to a concentration of four pounds of $FeCl_3$ per gallon, for operation convenience and protection against crystallization in cold weather. Ferric chloride is moved from storage to the various feeding points by means of compressed air.

All chemical solutions are carried in rubber hose lines that have proved very satisfactory for conveying both lime and ferric chloride. The contract cost of the changes and additions to the West Side plant was approximately $58,000.

The sludge incinerator installed at this time is a Nichols Herreshoff multiple hearth type, being 16 ft. 9 in. outside diameter and having 6 hearths. It has a rated capacity of 50 tons per day of sludge cake, containing 65 per cent moisture. Some of the heat in the exhaust gases is recovered by passing them through a combustion air preheater. The central shaft and rabble arms are air cooled, this air eventually forming combustion air. Auxiliary fuel is furnished in the form of fuel oil and is applied by means of four oil burners located on hearths No. 2 and No. 4. Although oil burners are provided on hearth No. 1, they are not used, as it has been found that the other burners give more economical results. (For a more complete description of the incinerator installed, reference is made to the article by Owen.[5]) The total cost of the sludge incinerator installed was $56,000.

Construction Costs Including PWA Contracts

The total construction cost of the East Side plant was $414,530 and the West Side plant $594,353, making a total investment in sewage treatment facilities of $1,008,883.

Loading and Results

The total population load of the East Side plant is estimated to be 155,000 at the present time. The flow for the last 12 months was 6,024 M.G. or an average rate of 16.5 M.G. per day. Sedimentation in the plant accounted for a reduction in suspended solids of 64 per cent, or 1,230 pounds per M.G., which represents a total of 7,415,000 pounds for the year. B.O.D. reduction amounted to 46 per cent or 489 pounds per M.G., or a total of 2,945,000 pounds for the year. These figures represent reductions per capita per year of 48 pounds of suspended solids and 19 pounds of B.O.D.

Table I is a tabulation of the flows and reductions through the plant by the month.

Table I

Date	Flow	B.O.D.			Susp. Sol.			B.O.D. Total 1000's Pounds	Rem. P.M.G.	S. S. Total 1000's Pounds	Rem. P.M.G.
		Raw P.P.M.	Effluent P.P.M.	% Red.	Raw P.P.M.	Effluent % Red.	% Red.				
1935											
June	606	169	81	52	351	73	78	446	740	1404	2320
July	483	188	89	53	385	88	77	398	825	1193	2470
August	426	142	83	42	216	78	64	209	491	488	1145
September	360	166	94	44	248	107	57	216	600	423	1173
October	374	155	83	46	227	95	58	224	600	412	1100
November	492	135	71	46	252	113	55	262	534	570	1158
December	536	139	70	50	204	76	62	309	575	527	1067
1936											
January	603	115	65	43	216	88	59	251	417	645	1067
February	550	102	60	59	148	61	59	193	350	399	725
March	580	91	57	37	211	74	65	164	283	662	1142
April	449	100	61	39	141	63	55	146	325	292	650
May	565	74	47	36	145	60	55	127	225	400	708
Total											
Average	6024	1576	861		2744	976		2945		7415	
		129*	70*	46*	220*	80*	64*		489*		1230*

* Weighted.

The West Side plant has a tributary population of approximately 22,000. The total flow for the past year was 917 M.G., or a daily average of 2.5 M.G. Suspended solids reduction in this plant amounted to 91 per cent, or 2,000 pounds per M.G., which represents a total for the year of 1,837,000 pounds, or 84 pounds per capita per year. B.O.D. reduction was 72 per cent, or 990 pounds per M.G., or a total for the

year of 907,000 pounds, which is equivalent to 41 pounds per capita per year. The total amount of chemicals used was 246,000 pounds of ferric chloride and 205,000 pounds of calcium oxide, which represents 268 pounds per M.G. and 224 pounds per M.G. respectively.

Table II is a more detailed setup of operating conditions and results at the West Side plant for the last twelve months.

TABLE II

Date	Flow M.G.	B.O.D.			Susp. Sol.			B.O.D. Total 1000's Lbs.	REM. P.M.G.	S.S. Total 1000's Lbs.	REM. P.M.G.	$FeCl_2$ P.M.G.	CaO P.M.G.
		Raw P.P.M.	Effluent P.P.M.	% Red.	Raw P.P.M.	Effluent P.P.M.	% Red.						
1935													
Jan.	101	141	37	73	191	21	89	87	860	144	1420	248	155
June	95	219	53	76	367	22	94	131	1380	271	2860	272	236
Aug.	85	195	36	82	460	27	94	112	1320	305	3590	327	300
Sept.	65	226	49	78	375	31	92	96	1470	187	2850	380	312
Oct.	61	201	46	77	356	33	91	79	1290	165	2710	412	440
Nov.	65	145	42	71	235	19	92	52	810	116	1790	264	200
Dec.	66	208	42	80	400	21	95	91	1380	207	3150	327	258
1936													
Jan.	72	187	48	69	272	25	91	66	910	149	2060	310	270
Feb.	48	205	55	73	222	26	88	59	1250	77	1630	368	325
March	69	93	41	56	120	28	76	30	430	53	770	197	112
April	90	100	31	69	90	23	74	52	580	50	560	146	120
May	100	98	35	66	158	22	86	52	520	113	1130	138	110
Total	917							907		1837			
Average		164*	42*	74*	265*	24*	91*		990*		2000*	268*	224*

* Weighted.
Total $FeCl_3$, 246,000.
Total CaO, 205,000.

TOTAL OPERATION COSTS

The operation, fixed, and total costs of the sewage treatment works, set up on various bases, are shown in Table III.

TABLE III

	Annual	Per M.G. Treated	Per Capita Served	Per 1000 Lbs. Solids Reduction	Per 1000 Lbs. B.O.D. Reduction	Per M.G. Per % Degree Treatment
Fixed	68,000.00	9.80	.38	7.35	17.67	.20
Operation	78,000.00	11.25	.44	8.33	20.30	.23
Total	146,000.00	21.05	.82	15.68	37.97	.43

These figures may be approximately divided so as to show the relative costs of treatment at the East Side plant and the West Side plant. However, the figures to be given are not intended to be used as exact, due to the fact that accurate calculation is well nigh impossible with the combination of sewage treatment and sludge disposal as it exists in Dearborn. The approximate cost per M.G. of sewage treated at the East Side plant is $10.15. Similarly, the cost at the West Side plant is $18.45. The total cost per million gallons at the East Side plant is approximately $19.95 and at the West Side plant $28.25. All of these figures include the cost of sludge disposal.

Operation—Sewage Treatment

The operation of the East Side plant is essentially that which one would find at any ordinary sedimentation plant. That is, it consists of sewage pumping, grit removal, the ordinary Imhoff tank operations, and sludge pumping. The latter is somewhat unusual probably because of the fact that the sludge must be pumped for a distance of about 4 miles and at a pressure of about 50 pounds per square inch. Sludge is pumped to the West Side plant at a rate usually of about 350,000 gallons per day and with a solids concentration varying from 2 to 8 per cent.

At the West Side plant the sewage is pumped to the mixing channels, previously described, where the chemicals are added, and the sewage is flocculated, settled, and passed through the magnetite filter. The use of lime is limited to adjustment of the alkalinity of the sewage to conditions of optimum flocculation, that is, it is not used in itself as a precipitating agent. Ferric chloride is the active precipitant. It has been found by experience that the best results are obtained when maintaining a pH in the effluent of 7.8 to 8.1. This range was determined from experience, as the range in which calcium carbonate would not be precipitated upon the sand and screens of the magnetite filters and the range in which ferric chloride would not be reduced to ferrous compounds. If the pH is allowed to go above the upper limit, the calcium carbonate deposit on the various parts of the equipment will cause a great deal of trouble, and if the pH is allowed to go much below the lower limit the ferric chloride is materially reduced to ferrous salts, including the ferrous sulphide, which gives a brackish appearance to the effluent.

Operation of the magnetite filters is relatively simple. Practically no difficulty is had with the cleaning mechanism. It is necessary to give some attention to the sand in the filter and wash it down with a hose stream. However, it is only necessary to do this at periods of from 2 to 6 months, and then it requires one man only about 4 to 5 hours to do the job. Chlorination is no longer practised at the West Side plant as it has been learned that the quality of effluent turned out without chlorination is sufficient to meet the requirements of the Rouge River.

Operation—Sludge Disposal

Of the two clarifiers provided at the West Side plant, only one is used for sewage treatment. The second clarifier is reserved for storage of the East Side Sludge. The sludge is pumped to this clarifier as required and is allowed to settle, and concentrate. From the center of the clarifier the concentrated sludge flows by gravity to the sump referred to in the beginning of this paper. The manner of taking the sludge from the sump and dosing it with chemicals for conditioning has already been described. The sludge from the West Side sewage, in the other clarifier, is removed and treated in a similar manner. The sludge flows from the mixers to the pans of the vacuum filters by

gravity. The level of sludge in the filter pan is maintained at a constant elevation, or nearly so, by manual control of a Reeves variable speed transmission, which controls the speed of the sludge elevator. Only one filter is used at a time since the installation of the sludge incinerator.

There are several prime requirements in the operation of the vacuum filters. First, the rate of production of sludge cake should be as uniform as possible in order to give a uniform rate of feed to the sludge incinerator. It is obvious, of course, that it is most desirable to have the lowest possible moisture content in the cake as discharged from the filter. The average amount of ferric chloride used for conditioning is 2.0 per cent of the dry solids contained in the sludge. Similarly, the average amount of calcium oxide used for conditioning is 11.8 per cent.

From the filter the sludge discharges onto a belt conveyor which carries it over a weightometer and to the feed hopper on the incinerator. The weightometer indicates, records, and totalizes the amount of sludge passing over it. It is generally necessary to stop the vacuum filter and run the belt empty over the weightometer about once a shift in order to establish a tare weight on the belt. The tare weight of the belt will vary somewhat due to material sticking to the face of it.

The sludge incinerator is manually controlled throughout. Although provision was made for the control of one oil burner automatically, it has been found satisfactory and desirable to eliminate the use of the automatic feature.

The amount of combustion air furnished to the incinerator is also controlled manually, as is the draft which is maintained by a fan in the exhaust gas duct. Ordinarily, the sludge deposited in the furnace will dry on the two top hearths; it will ignite on the third; and continue to burn on the fourth, and possibly some on the fifth; but generally on the fifth and sixth hearths there is only ash, which is cooled to some extent by the incoming combustion air. At the same time the air recieves some additional preheating. The usual exhaust gas temperature maintained in the incinerator is from 800° to 1000° F.

The question may arise as to odor production at these relatively low temperatures. However, obnoxious odor is not discharged from the stack. A probable answer to this is the fact that the material on the top part will be relatively cool, in fact, so cool that odor producing volatiles are probably not distilled off. A test of this condition was made by taking samples at various points along the path of travel for the sludge on the top hearth and determining the temperature of those samples when the hearth was operating at a temperature of about 850° F. It was found that the evaporation of moisture from the sludge was sufficiently rapid that the maximum temperature of the material, which obtained at the point where the sludge was ready to drop on to the second hearth, was but 152° F.

The ash from the furnace is continuously discharged into the boot

of a bucket elevator, which conveys it to an outside storage hopper. Periodically, the ash is removed from the storage hopper and transferred, by truck, to a dump. The ash is quite dusty but once it has become moist it cakes quite well and, consequently, does not blow thereafter.

In the operation of the incinerator it has been found most practical and economical to operate the furnace at or near capacity rate while sludge is available and then to shut down the furnace, just burning enough oil to keep it at an average temperature of about 600° F., until such time as sufficient sludge has accumulated to warrant starting operation again on a full capacity schedule. The labor, power, and fuel requirements are all lower when the furnace is operated in this manner than with continuous operation at partial capacity.

TABLE IV.—*Sludge Filtration and Incineration*
Quantities and Costs of Operation

Item	Quantity	Unit Cost	Total Cost
Sludge Filtration			
Operation	5,276 hours		
Sludge Cake	12,018 tons		
Dry Solids	4,074 tons		
Calcium Oxide	813,000 pounds	.003	$2,439.00
Ferric Chloride	137,000 pounds	.018	2,466.00
Power	105,500 K.W.H.	.01	1,055.00
Labor	2,638 hours	.80	2,110.40
Laboratory	450 hours	.90	405.00
Supervision	400 hours	1.10	440.00
Total			$8,915.40
Sludge Incineration			
Operation	5,276 hours		
Sludge Cake	12,018 tons		
Dry Solids	4,074 tons		
Moisture	7,944 tons		
Fuel Oil	21,508 gal.	.045	$ 967.86
Power	46,295 K.W.H.	.01	462.95
Labor	2,638 hours	.80	2,110.40
Laboratory	225 hours	.90	202.50
Supervision	450 hours	1.10	495.00
Total			$4,238.71
Idle Time	3,484 hours		
Fuel Oil	16,014 gal.	.045	$ 720.63
Power	13,931 K.W.H.	.01	139.31
Labor	871 hours	.80	696.80
Total			$1,556.74
Ash Disposal			
Labor	780 hours	.80	$ 624.00
Truck			300.00
Total			$ 924.00

Table IV shows the quantities of materials handled and the costs of operation for sludge disposal for both plants for the past 12 months. Labor for filter and incinerator operation is established as one-half hour of labor for each unit per hour of operation. This matter of labor requirements was studied for several months, at the end of which time the figures given were deduced. Dearborn had the advantage of an unusually low ferric chloride cost during the last year, so, in order to make the tabulations more comparable with present conditions, the present contract price of ferric chloride, which is approximately $1.80 per cwt. of anhydrous, is used.

The fuel oil actually used in sludge incineration was 1.8 gallons per ton of sludge cake, and the power was 3.8 K.W.H.

In order that the table may be useful for cost comparison purposes all items have been given unit prices. Ash disposal, although it is an inherent part of sludge disposal as a whole rather than incineration alone, is included under the latter, but has been segregated in this table and Table VII as well. In Dearborn a six mile round trip haul is required to convey the ash to a dump.

Table V giving maintenance costs covers the actual expenditures for supplies and what is believed to be a reasonably accurate estimate of the labor necessary in maintenance work.

TABLE V.—*Sludge Filtration and Incineration—Maintenance Costs*

Item	Supplies	Labor	Total
Sludge Filtration	$330.00	$1,320.00	$1,650.00
Sludge Incineration	33.50	544.00	577.50
Total	$363.50	$1,864.00	$2,227.50

In Table VI the only fixed charges that have been applied as such are the actual costs of amortizing the bonded indebtedness necessary for the construction of the items. No interest on investment or similar charges have been included. The cost of the building has not been included in these fixed charges in that building costs will vary widely with respect to type and locality.

TABLE VI.—*Sludge Filtration and Incineration—Fixed Costs*

Cost of Filters and Appurtenances	$24,000.00	
Amortization at 7.6%		$1,824.00
Cost of Incinerator	$56,000.00	
Amortization at 5.8%		$3,248.00
Total Annual Cost		$5,072.00

Table VII is a summary of the total costs of sludge disposal to the city of Dearborn, set up on various bases. The cost per million gallons at the East plant and the West plant are necessarily prorated figures. If the costs be prorated to the basis of continuous, full capacity operation, the cost per ton of dry solids incinerated becomes $4.65.

According to Jones,[6] the total cost of disposing of sludge, including

interest and depreciation, on covered sand drying beds is about $8.00 to $9.00 per ton of dry solids. If we raise the cost of filtering and incinerating one ton of dry solids, on the basis of continuous, full capacity operation, to $5.65 (to cover the fixed charges on a building), the cost is still materially below covered sand drying costs.

TABLE VII.—*Sludge Filtration and Incineration—Total Costs*

Item	Total Cost	Per Ton of Sludge Cake	Per Ton of Dry Solids	Per M.G. of Sewage	
				East Plant *	West Plant *
Sludge Filtration					
Operation	$ 8,915.40	.74	2.19	1.15	2.15
Maintenance	1,650.00	.14	.40	.22	.40
Fixed	1,824.00	.15	.45	.24	.44
Total	$12,389.40	1.03	3.04	1.61	2.99
Sludge Incineration					
Operation	$ 4,238.71	.35	1.04	.58	1.07
Maintenance	577.50	.05	.14	.07	.14
Fixed	3,248.00	.27	.80	.42	.78
Total	$ 8,064.21	.67	1.98	1.07	1.99
Idle Time	$ 1,556.74	.13	.38	.20	.37
Ash Disposal	924.00	.08	.23	.12	.22
Total Disposal Costs	$23,034.35	1.91	5.63	3.00	5.57

* Prorated Values.

CONCLUSIONS

The sewage treatment plants at Dearborn take care of the sewage treatment problem in a manner that is perfectly satisfactory for their location. It is believed that the costs of operation are economical for the degree of purification achieved. Sludge disposal by means of vacuum filtration and incineration is highly satisfactory and exceptionally economical.

Acknowledgment is made of the courtesy of Clark M. Greene, Superintendent of Public Works and Engineering in Dearborn, in permitting this paper to be presented and to C. Austin Habermehl, who is in charge of sewage treatment operations, for furnishing monthly reports and other data.

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CONSTRUCTION OF SEWERS AND SEWAGE TREATMENT WORKS IN PENNSYLVANIA UNDER WORKS PROGRESS ADMINISTRATION *

By Paul L. A. Keiser

Pottstown, Pa.

The sewer work in the borough of Ambler comprises a complete sanitary sewerage system and a sewage treatment plant, all of which is being carried on with government financial aid under the two Federal Emergency Employment Relief Agencies, namely W.P.A. (Works Progress Administration) and P.W.A. (Public Works Administration).

The entire system as laid out by the designing engineers, Albright & Friel, Inc., of Philadelphia, Pa., is divided into four districts. That part of the work known as and included in District No. 1, comprising approximately 5.2 miles of sewers, together with the construction of the complete sewage treatment works, is the portion of the work now being completed under the supervision of W.P.A. The remainder of the sewerage system in Districts No. 2, 3 and 4, comprising a total of approximately 14.23 miles of sewers, is being constructed at the present time by contract under the supervision of P.W.A.

The Ambler sewage treatment works is an activated sludge type plant with mechanical aeration, designed for a population of 4500 people. It consists of a screen chamber, two primary settling tanks equipped with mechanical collectors, a pumping station, a battery of six aeration tanks, final settling tanks provided with mechanical collectors, a chlorine contact chamber, a sludge conditioning tank, two sludge digestion tanks with provisions for sludge heating and gas collection, a steel gas holder, and a glass covered sludge drying bed.

The sewage from the entire municipality flows by gravity to the site of the treatment works and at this point is pumped through the plant. The sewage pumping station is constructed as a unit of the plant and serves also to house the sludge pumping and chlorination facilities. Chemical laboratory equipment, a toilet, activated sludge control facilities, and other miscellaneous facilities common to this type of plant, are also housed in the pumping station.

The pump house is a three story, concrete structure. The lower floor contains a sewage suction well divided into two compartments by a wall, and a dry well adjacent thereto for housing the vertical centrifugal sewage pumps. A gas fired boiler for heating the building is also located in the pump room. On the second or middle floor of the building are located the vertical motors for driving the two sewage pumps, and also the motor control equipment. The two sewage pumps are the variable speed type, each ranging in capacity, by four steps,

* Presented before the Tenth Annual Conference of the Pennsylvania Sewage Works Association, State College, June 24, 1936.

from 125 G.P.M. to 500 G.P.M. They operate automatically with the rise and fall of the sewage level in the wet well.

On the middle floor are also located two 75 G.P.M. sludge pumps for returning activated sludge from the final settling tanks to either the aeration tanks or the digestion tanks. One of these pumps, which is of the plunger type, is of constant speed, while the other may be varied in capacity from 10 G.P.M. to 75 G.P.M. Each pump is equipped with a counter meter for approximating the quantities discharged from it.

The upper, or main floor of the station is used for housing the two 150 G.P.M. raw sludge pumps and other miscellaneous facilities. A built-in laboratory analysis cabinet, a toilet room, a chlorination room and a separate room for the activated sludge division box, have been provided on this floor. Flush type electrical power and lighting control panels are located on the main floor. The superstructure is of colonial design with red brick and limestone trim. The roof is of Ambler asbestos shingles and the interior is lined with a buff tone salt glazed bricktile.

Both the primary and final settling tanks are provided with straight-line, mechanical sludge collectors. The six aeration tanks are equipped with mechanical aerators of the spinner type. The tanks and channels of these aerators are arranged so that flexibility may be obtained in operation by changing the direction of flow or by by-passing any tank. The sludge conditioning tank is a hopper-bottomed structure into which sludge may be drawn from the digestors or aerators for lime or other treatment. After leaving the chlorination contact chamber, which is a rectangular baffled basin, the effluent flows over a weir before discharging into Wissahickon Creek. A gauge mounted on the wall in the pump house records the flow of sewage over the effluent weir.

The sludge digestion tanks are duplicate, hopper bottomed, square units with fixed concrete tops. Means are provided for drawing off the supernatant liquor from the tanks at various levels. Gas is collected in oil sealed, cast iron domes and piped to a gas holder.

The pipe gallery adjacent to the digestors is provided with a separate heater room containing a gas fired hot water boiler, hot water circulating pumps, sludge temperature recording gauges, and a combined condensate, pressure relief and flame trap. An excess gas burner is located outside of the boiler room.

The sludge is drawn by gravity from the digestion tanks to the sludge drying bed. This bed is covered with a standard green house enclosure. The effluent from the sludge bed drains by gravity to the wet well of the sewage pumping station.

The sanitary sewers in District No. 1, constructed mainly with relief labor under W.P.A., comprise approximately 24,500 lineal feet of 8 in. to 12 in. sewers and 3000 lineal feet of 6 in. laterals.

The work in this area was started under L.W.D. (Local Works Division) October 18, 1934 and continued under a field organization, separate from the sewage treatment works organization, until June

30, 1935, the close of the fiscal year of the L.W.D. Up to that time all but 1500 feet of the sewers in District No. 1 had been completed. The remainder of the lines in District No. 1 were not constructed until work was resumed in November, 1935, under the new W.P.A. set-up. At that time both the sewers and the plant were placed under the supervision of one field organization, which proved to be a more desirable plan than the one tried under L.W.D.

The subsurface conditions found in this District were somewhat worse than had been anticipated, due mainly to the large quantity of rock and water encountered. These conditions would not have been so serious with a trained, experienced, and an aggressive personnel, such as is generally found in a reliable contractor's organization, but they did prove costly and difficult to the organization that grew out of this relief exigency.

An interesting achievement of the untrained common labor personnel of this project was the construction of a tunnel crossover for a 12 in. outfall sewer extending from a corner of the borough limits through a swamp to the treatment plant. The crossover was located approximately 9 feet under a four track electrified branch of the Reading Railroad. It was constructed by jacking a 36 in. corrugated, galvanized steel pipe in 10 ft. sections through the railroad embankment for a total distance of approximately 70 feet. As the jacking operation continued (using a 100 ton hydraulic jack for applying pressure to the corrugated steel shell) the earth within the area of the cover pipe was removed by tunnel "muckers" using pneumatic picks.

Construction of the sewage treatment works was started in November 1934, shortly after the sewers in District No. 1 had been begun. Many annoying and costly difficulties were encountered shortly after the commencement of this phase of the work, due to an improperly organized field administrative staff. About the 15th of January 1935 a reorganization of the field supervision of the project was effected through the splendid cooperation of the County Administrator of Work Relief and the designing engineers. The field administrative staff was then set up to as nearly as possible emulate a general contractor's organization, together with the usual line-up of sub-contractors' organizations.

The federal staff included a superintendent, general foreman, carpenter foreman, mechanical foreman, electrical foreman, plumbing foreman, labor foreman, two timekeepers, a storekeeper, and a master mechanic for construction equipment.

The designing engineers supplied a supervising field engineer and the borough supplied a clerk of works, who acted as a purchasing agent for minor purchases only. All the major purchases of construction materials and plant equipment were made by contract after detailed bills of material had been made up by the engineers and requests for bids published by the borough council.

A list of required construction equipment was planned and submitted for approval. Most of it was supplied on rental contracts by

the federal agencies. The following list will give some idea of the major pieces of equipment furnished for the treatment works only that were considered necessary for a project such as this and which up to this time had not been supplied.

2 Air compressors with jack hammers, paving breakers, clay picks, tampers, and chipping hammers.
2 Two-bag concrete mixers.
2 Self-priming centrifugal pumps (one 3 in. pump), (one 2 in. pump).
1 4 in. diaphragm pump.
1 Force pump, for water supply.
2 Pneumatic concrete vibrators.
A complete kit of plumbers and electricians tools, vises, etc. Oil heaters for concrete mixers, oil hot water heaters, thawing torches, and material heating salamanders.

The government's formal application for the project called for the borough to supply *all* materials, all small tools, and a few of the larger pieces of equipment, such as a trenching machine, compressor and several pumps. The government was to supply all labor and some major pieces of rented equipment. Although the federal allowance for equipment rentals was supposed to be limited to 5 per cent of the cost of the labor contribution, this amount of necessity was materially increased as the job progressed.

The cost to the borough to date for small tools alone has been approximately $3500. A large portion of this excessive cost was due to the heavy loss of small tools and miscellaneous equipment during the first few months of operation, before a tool control system had been installed. As an example, during the period from October, 1934, to January, 1935 (3 months), the borough lost over 100 pairs of rubber boots, 90 pairs of goggles and innumerable wrenches, wrecking bars, hammers, chisels, shovels, etc. As soon as a storekeeper was placed on the job and the tools passed out to the workmen on a check system all losses except from breakage were practically eliminated.

It is interesting to note here that it was found expedient to revise the method of distributing gasoline and oil and to give this phase of the job most careful supervision. The method used was to place the treatment works storekeeper in charge of its distribution on both the sewer and plant projects and to have the person who received gasoline or oil sign a triplicate receipt. These signed receipts were presented daily to the superintendent of construction of the treatment works who also signed them, marked approved. In spite of the care with which this item was handled by the revised organization, the cost to the borough to date for gasoline and oil has been approximately $4,350.

In the construction of a project of this kind under emergency relief conditions, it was natural to expect that we would encounter a number of problems. Chief among these problems were:

The reestablishment of confidence between the borough council and the new field organization.

The benefit of this reestablishment was of utmost importance, for where there is a lack of confidence or a lack of co-operation there are bound to be costly delays. The handling of this problem was one of the most important elements in the ultimate success of the job.

Another problem was the establishment of healthful job discipline, contentment among the workers and a desire among the mechanics to do better than ordinary work. In other words, the thing that was necessary was "esprit de corps," or the creation of a loyalty to the job in hand. This spirit is just as essential in the construction industry as in a team that sets out to win a football game. This was accomplished through discussions at mass meetings of the workmen, held on the job. The resulting teamwork had the same effect on our W.P.A. job as the loyalty among the workmen of the usual contractors organization has in private industry. A group of men, no matter what the situation may be, under proper leadership, can be organized for teamwork if the proper stimulus is given them.

It is important to catch the significance of the dates during which most of this work was done. Work on the plant was started November, 1934, and shut down the end of June, 1935; resumed November, 1935; and again shut down June 24, 1936, just within sight of completion. Practically all of this time, with the exception of a few months, was during the bad weather period.

The winter months being the period of the year when relief from human suffering is most needed, our job consequently was overloaded with manpower. As we approached each spring and the good weather of the summer months came on, the need for relief eased, and jobs were cut to conserve funds.

As an example of the costliness of the overloading of manpower in the severe winter months, it is interesting to note that the amount of money spent on the sewage treatment works project alone, for non-productive hours (time for which the men were paid but for which they actually did not work) during this past winter, totaled approximately twenty five thousand dollars ($25,000). During the previous winter it was approximately $15,000, making a total of $40,000.

Outstanding among the difficult problems encountered in W.P.A. work, is the inability to hold to a budgeted work schedule or a progress schedule. As an example, in a contractual arrangement, our objective always is to accomplish the greatest amount of work in the shortest possible time, and each phase of the job is carefully planned and scheduled.

In other words, you know from day to day before it occurs, how much work will be accomplished and how you are progressing towards final completion. In the relief set-up for instance, where carpenters, cement finishers, plumbers, electricians, and painters, etc., work but 85 hours out of a possible 130 hours per month (about 11 days) and where bricklayers and steel workers are limited to 60 hours per month (about 8 days), you can appreciate it always will be difficult to make the desired progress at the desired cost where these maximum hour limita-

Ambler Sewerage System—Comparative Cost Data
June 19, 1936

District Number	No. 1		No. 2	No. 3	No. 4	No. 2, No. 3 and No. 4 Combined
Federal Project Under	W.P.A.	Federal Project Under	P.W.A.	P.W.A.	P.W.A.	
Size and Type of Sewer	8 In. to 12 In. V.P.	Size and Type of Sewer	8 In. to 12 In. V.P. 16 In. to 18 In. C.I.	8 In. to 12 In. V.P. 8 In. to 12 In. C.I.	8 In. to 12 In. V.P. 8 In. to 12 in. C.I.	
Length of Sewer, Lin. Ft.	24,500	Length of Sewer, Lin. Ft.	15,658	17,458	34,250	
Length of Laterals, Lin. Ft.	3,000	Length of Laterals, Lin. Ft.	1,400	3,600	2,800	
Combined Length, Lin. Ft.	27,500	Combined Length, Lin. Ft.	17,058	21,058	37,050	75,166
Total Borough Contribution	$ 36,494.33	Contract Prices	$67,875.75	$62,898.40	$64,017.00	
Total Federal Contribution	$242,433.03	Total Contracts 2, 3 and 4				$194,791.15
		Total Eng., Legal. Int., Adv., Contingencies, Etc.				$ 20,039.45
Total Borough and Federal Contribution	$278,927.36	Grand Total				$214,830.60
		Total Borough Contribution (55%)				$118,156.83
		Total Federal Contribution (45%)				$ 96,673.77
Borough Cost Per Lin. Ft.	1.327	Borough Cost Per Lin. Ft.				1.572
Federal Cost per Lin. Ft.	8.815	Federal Cost per Lin. Ft.				1.286
Combined Borough and Federal Cost Per Lin. Ft.	10.142	Combined Borough and Federal Cost Per Lin. Ft.				2.858

Ambler Sewage Treatment Works—Comparative Cost Data

Federal Project Under	W.P.A.	
Total Borough Contribution	$ 79,533.26	
Total Federal Contribution	114,460.59	
Total Borough and Federal Contribution	$193,993.85	

Summary

Total Borough Contribution Sewers, Districts 1 to 4, Under W.P.A. and P.W.A.	$154,651.16	
Total Borough Contribution Sewage Treatment Works	79,533.26	
Total Borough Contribution Sewers and Sewage Treatment Works		$234,184.42
Total Federal Contribution Sewers, Districts 1 to 4, Under W.P.A. and P.W.A.	$339,106.80	
Total Federal Contribution Sewage Treatment Works	114,460.59	
Total Federal Contribution Sewers and Sewage Treatment Works		$453,567.39
Grand Total, Complete Sewerage System		$687,751.81

tions are maintained. As carefully as we attempted to arrange the staggering of our mechanics' time so as to make each phase of our operation continue without interruption, we frequently found ourselves faced by a shortage of mechanics when we needed them most, or inversely, the job would be overloaded because of some slip in our plans due to bad weather or some other delaying factor. The inflexibility of these labor regulations adds materially to the difficulties encountered during the construction of any project as complex as a sewage treatment works.

Some interesting comparative cost figures can be given on the sewer work, particularly, because District No. 1 done under L.W.D. and W.P.A., is now completed and costs on Districts Nos. 2, 3 and 4 have been fixed by separate contract under P.W.A. These comparative costs, including all overhead charges, such as engineering and legal fees, interest, advertising, contingencies, etc., are as shown in table on page 825.

In conclusion, my observation during construction on a project of this kind is, that there are two philosophies working in opposition. The one is a philosophy of a little work for all, regardless of quality or cost; the other is the philosophy of the best work by those who are qualified to give the best in the most efficient manner.

Our chief problem here was to merge these two philosophies in spite of their conflict, in order to get the job done as well as practical.

Discussion

By Robert Hall Craig

Consulting Sanitary and Hydraulic Engineer, Harrisburg, Pa.

In discussing the subject "Construction of a Sewage Treatment Plant and Sewer System under W.P.A." as covered so thoroughly by Mr. Keiser of Pottstown in his paper, I feel that the Pennsylvania Sewage Works Association should place on the record some description of the works relief agencies which are administering to those in need of relief during this depression period. If it is not now clearly recorded, we will miss the opportunity. Within another 10 years we will be looking back upon this economic depression, as something in the dim distant past as we are already looking back upon the world war period.

In the first place let me clearly state that I feel that the construction of sewers is the ideal type of works relief project; either under W.P.A. or under P.W.A. The swinging of a pick in a trench and the throwing out of dirt with a shovel are the easiest types of labor, yet their results—the construction of a sewer system—bring the greatest amount of good to a community.

We in Pennsylvania have not been able to grasp the full opportunity presented by these National Works Relief plans, chiefly because of the necessity of going to the municipal taxpayers for the approval of bond issues, coupled with the natural reluctance of municipal authorities to

ask political constituents to increase municipal indebtedness during a period of depression. Under the P.W.A. program, approximately 80 per cent of the monies spent for construction projects went for public schools in the areas outside of Philadelphia and Pittsburgh. This was chiefly because of the well knit together divisions of the State Department of Public Instruction, acting through the central office at Harrisburg.

The Pennsylvania Department of Public Instruction is probably one of the greatest and most closely controlled bureaucracies in the United States. It quickly organized the school districts, which in turn prepared plans for new school buildings and asked the authority of the taxpayers to issue bonds. There is no such organization when it comes to boroughs and cities. Our own Department of Health did everything in its power to persuade municipalities to take full advantage of these monies being given by the Federal Government for use in sewer construction.

The head of the Public Works Administration at Washington is Mr. Harold L. Ickes, a native Pennsylvanian; the present Pennsylvania Director, Mr. G. Douglas Andrews, was a consulting engineer, taken from private practice, who had at one time been an engineer in the State Department of Health. The Administrator of W.P.A. in Washington is Mr. Harry L. Hopkins and the Pennsylvania State Director is Mr. Edward N. Jones.

Basic differences in methods of functioning of P.W.A. (Public Works Administration—Mr. Ickes) and W.P.A. (Works Progress Administration—Mr. Hopkins) are these:

P.W.A. (Public Works Administration)

1. Promotion of project may be by political subdivision or private corporation; vast majority are municipal.
2. Size of project practically unlimited.
3. Money provided by 45 per cent Federal Gift and 55 per cent Federal Loan, if desired. Interest rate 4 per cent, loan amortized over life of structures, not to exceed 30 years.
4. Method of performing work is by public letting to contractor on basis of lowest bid; sponsor awards contract under P.W.A. supervision.
5. Certainty of completion of project is assured once loan and grant are approved and money is earmarked.
6. Time for submitting projects has closed under present appropriations. Jobs were to be under contract by December 15, 1935.
7. Workers must be obtained from State and Federal Employment agencies in so far as possible. Skilled workers may be brought in.
8. Rates of pay cannot be less than fair average of general rate schedule in area affected.
9. Design and supervision of construction is by competent engineers or architects in private practice, or in sponsor's employ.

W.P.A. (Works Progress Administration)

1. Promotion of project is by political subdivision; majority are municipal.
2. Size of project originally limited to $25,000 of Federal money plus local funds for materials, tools, etc., has now been liberalized. Second project may be continuation of first.
3. Money is provided by Federal Government for labor and by municipality for materials, tools, equipment, technical supervision.
4. Method of performing work is by "force account."
5. Certainty of completion is problematical. Funds are earmarked but there is always doubt as to continuation of general program of this type of work relief.
6. Time for submitting projects is continuous, though municipalities should not delay applications.
7. Workers must be obtained from local relief rolls, and attempt is made to utilize a man's qualifications to limited extent.
8. Rates of pay are fixed at average common labor rates per hour and worker is limited to fixed monthly income.
9. Design and supervision may be by W.P.A. engineers or by consultants or municipal engineers, depending on type of project.

There has been much criticism of waste and inefficiency both by the engineering profession and by the general public. I hold no brief, for either waste or extravagance. To an engineer those two evils are abhorrent. I do say, however, that from the standpoint of a municipality spending the taxpayer's dollar, that if such municipality can construct a sewer system, because of funds given to it from the Federal government, at a less "out of pocket" cost than if such municipality were forced to build the same sewer system fully with its own funds, then, regardless of the waste or inefficiency of relief labor, the administrators of that municipality have rendered a splendid public service. This assumes that the final construction work is equal to normal contract work.

So that I may not be misunderstood, I state the same proposition in another manner. If a municipal administration can take advantage of the fact that the Federal government will pay for a large share of the cost of a sewer system in that municipality, then so long as the local taxpayer pays a less amount than he would have to pay if the municipality constructed that system entirely with its own funds, the municipal authorities have rendered a splendid public service.

Let us continue to promote sewer construction as an ideal relief project so long as works relief continues.

COMPREHENSIVE SEWAGE DISPOSAL PLAN OF NASSAU COUNTY *

By Clarence MacCallum

Consulting Sanitary Engineer, Nassau County Sanitation Commission

Nassau County, like many similar areas, sooner or later develops a major problem in the proper collection and disposal of sanitary sewage. The county has an area of approximately 275 square miles and a population of 361,000 persons. It has in the past thirty years been the object of marked increase in population. For instance, in the decade from 1920 to 1930, the population increased 140 per cent. This is no doubt due to the fact that it is a part of the metropolitan area.

The first definite step towards solution of the sewage disposal problem came in 1930 when the Governor appointed a commission for the purpose of making a preliminary study. This commission was headed by Dr. Parran, Commissioner of the State Department of Health. As a result of the painstaking care of the Commission and the engineers of the State Department of Health, a report was issued in 1931 which indicated beyond a doubt the necessity for going into the matter much more extensively. The report recommended legislation to make this complete study possible.

In 1934 the necessary legislation was passed authorizing a comprehensive sanitary survey to ascertain the best method of solving the problem from an engineering and health standpoint; to ascertain what could be done with the present village sewer systems, and what part the county should take in a broad, far-sighted plan to care for future needs in the most economical manner and in conformity with good health standards.

One of the first steps was to ascertain where in the county the present population lived. This survey indicated that 95 per cent lived on 44 per cent of the area. To carry this essential data still further necessitated prophesying what the population would be thirty years hence by decades and where they would probably live.

Information was gathered from various sources: from the United States Census Bureau, from the New York Regional Plan Association, from a recent census of the P.W.A. and lastly from our own intimate knowledge of the county. The New York Regional Plan Association estimated 912,000 persons by 1965. The report of the State Department of Health in 1931 indicated a somewhat higher figure; about 1,400,000 persons by 1965. Our own estimates were based on the allocation of population densities per acre for all of the villages and unincorporated areas in the county, and when these latter figures were totaled up it made our estimate 1,180,000 persons by 1965 and this is the figure

* Presented at the Spring Meeting of the New York State Sewage Works Association, Lido Beach, Long Island, June 11–13, 1936.

used in the report as it was approximately an average of the New York Regional Plan estimate and that of the State Department of Health.

This means an increase of nearly 800,000 persons during the next thirty years. A further analysis of these figures indicated that the same 44 per cent of the area of the county would have between 95 per cent and 96 per cent of the total population in 1945, 1955 and 1965. The remaining 56 per cent of the county had a population density well below two persons per acre on the 1965 basis. We therefore eliminated this area from the problem. Its attempted solution would be too hazardous at this time.

There are now in the county sixteen sewage disposal plants and sewage systems, either under control of incorporated villages or special sewer districts. There are four special sewer districts on the north side of the county and one on the south side. The south side is the most populous and has the largest villages, therefore, the independent sewage systems are under their control in this part of the county. The present sewer facilities provide service for less than 200,000 of the 361,000 population, the balance are still on cesspools.

Having established the fact that but 44 per cent of the area of the county would need sewer facilities during the next thirty years, this area was divided into county sewer districts, each of these being the largest area of land that could economically be put in one district. There are fourteen of these districts in all. (See Figure.)

There are certain functions that certain units of government can perform just as well or better than other units. Considering this fact it was believed that the county should undertake no larger part of the program than was necessary. The villages and special sewer districts could without doubt install the laterals in their own streets and operate them just as well as the county could. The hygienic and economical disposal of sewage when once collected in the village was their serious problem and it is at this point that the county system starts.

Considering the individual village disposal plant from a cost standpoint we find that the history of the sixteen plants now in the county indicates an excessive first cost. The average is approximately $195,000 per one million gallons capacity per 24 hours. The plants vary in type from Imhoff tanks to activated sludge and the high cost is no doubt due to the fact that they are small in size and that numerous small additions to them produce a cost out of all proportion to the increased capacity. Should the individual plant system be pursued we would have 65 to 70 plants in the county by 1965.

From a hygienic standpoint this multiplicity of plants discharging into the ground waters, into fresh water streams and into the salt water bays would no doubt in time produce evils that would be extremely difficult to control.

With the present sixteen plants and with a population of 361,000 persons we now have marked signs of pollution in many of the salt water bays and harbors. With 800,000 additional people coming into

the county and with an increased number of disposal plants it is not difficult to prophesy what the condition of these bays might be in 1965.

The water supply of Nassau County comes entirely from rainfall, seeping through to the ground reservoir from which it is drawn through wells. There are no underground rivers feeding an inexhaustible supply of water to the county. Plans are now under way for capturing all of the rainfall that can be captured and returning it to the ground. Indications are that by 1970 if the population increases as anticipated the county will have reached the limit of its water supply.

Many of the villages of the county are located inland with no opportunity of discharging sewage effluent into stream or tidewater and for that reason are dependent upon seepage beds. With increased population more and more fresh water wells will be required and more and more sewage disposal plants discharging into ground waters, with the ultimate result that sewage plants and potable water supplies will draw closer and closer together. Should pollution of the beaches and the water supply continue to a point where the former are no longer usable with safety for bathing and other recreational purposes, and the latter require constant supervision and chlorination there is little doubt that all of the property values in the county would suffer, which would affect population, business and all of the other factors that are conducive to prosperity in the community. We have but to look at the counties to the west of us to gain some idea of the magnitude of a problem that remained unsolved for too long a period of time.

Should the method be pursued to the point where 65 or 70 independent sewage systems were built in the county we would be faced some day with the problem of trying to tie them together into one system. The cost would be much in excess of a well-planned county system put into operation at an early date.

The county system as previously mentioned divides 44 per cent of the area of the county into fourteen sewer districts, each of which comprises a trunk collecting system serving every village and unincorporated area within a district and providing each district with a sewage disposal plant.

In other words, there will be fourteen trunk networks and fourteen disposal plants. Each village or special sewer district will bring its sewage to a low point in the village where it will be pumped into the county trunk and from there flow by gravity to the plant located at tidewater.

With the firm conviction that the best interest of the county can be served by maintaining the bays and beaches free from pollution we have tentatively selected the chemical precipitation type of plant for the reason that it permits flexibility, is not excessive in first cost, its operating cost is not excessive and it accomplishes a high removal of suspended solids and B.O.D. We are well aware of the fact that the salt water bays provide more than ample dilution ratios for many years to come but we believe that it is to the best interest of all concerned that the effluent from the county plants be of the best from the start. We

are seeking the removal of at least 90 per cent of the suspended solids and 80 per cent of the B.O.D.

The taxpayers in any community are vitally interested in knowing what any major undertaking such as this might add to their tax bills. They are more concerned about how much it is going to cost than the benefits they will receive from a health standpoint. We, therefore, carried through in each of the fourteen districts a cost comparison, comparing the individual village system with the county wide collection and disposal system. The results for the county scheme are very gratifying from a cost standpoint.

If in the fourteen county sewer districts each village was permitted to pursue its own course, the villages during the next thirty-year period would have a capital outlay of about $27,400,000. This money is solely for disposal plants and the land upon which they would be built. To provide these same areas with county trunks and sewage disposal plants and the land upon which they would be built, would cost approximately $13,300,000, a capital outlay of less than one-half for the county system as against the village system.

In the case of the village system each village would have to pledge its own credit to carry out the work, and in the second instance the county would pledge its credit to carry out the work. Carrying this matter of cost still further, it is estimated that under the village system the interest, amortization and all operating costs would approximate $58,000,000 for a thirty year period. Whereas under the county system the amount that these same villages would have to pay would be approximately $34,600,000. The economy in first cost, as well as in operating cost, is quite evident from these figures.

In preparing the report for the Nassau County Sanitation Commission we had to satisfy many different public and private bodies. The State Department of Health was vitally interested in the engineering and health factors; the Long Island State Park Commission was concerned that nothing be done that would in any way damage the enormous investments that they have in Nassau County in the form of public beaches and parks; the Conservation Department of the State of New York was naturally concerned in the pollution of the bays and the effect it might have on the shellfish; the political bodies were concerned that nothing be done that would disturb the political equilibrium; and, last but not least, the taxpayers were interested to know just how it might affect his or her particular piece of property from a tax standpoint.

Each of the fourteen county districts has, within it, areas that are not now heavily populated, and although it may be ten or fifteen years before some of these areas are provided with laterals, common sense engineering dictates that sufficient trunk capacity be available for them when they need it. The trunk sewers are designed on the basis of the 1965 population allowing 250 gallons per capita per day for trunks carrying less than ten million gallons per day, and on the basis of 200 gallons per capita for trunks carrying more than ten million gallons

per day and as the average peak contribution per capita in the present sewer systems of the county is 145 gallons per day, it is believed that on the average the trunks will care for the population forty to forty-five years hence. Plants are designed on the basis of the estimated 1950 population thus permitting of additions conforming to advancement in the art.

As parts of each district will not require sewer service immediately it was believed that some differential tax rate should be applied so that those who use the system will pay more than those who do not use it even though the services are available to all from the beginning.

Each of the fourteen districts stand on its own feet financially and physically. Each district is established independent of all the others by procedure fixed by law, so that if any one of the fourteen districts is established only the property within that district is affected. The interest and amortization on the district bonds are levied against all property within that district. In other words, an *ad valorem* tax.

The costs of operation, maintenance, power, chemicals and all other items are levied against the particular village or special sewer district using the system in proportion to the number of millions of gallons that each discharges into the trunk system as measured by Venturi meter. In this manner, those who have facilities available for their use but who do not use them pay their share of the construction cost and those who use the system pay an additional amount in proportion to the usage they make of it.

To indicate to the taxpayer what the tax would be on his particular piece of property, tables were worked out by five-year periods for each of the fourteen districts, and assuming thirty year bonds at 4 per cent the average annual tax per $100 assessed valuation over the thirty-year period is 7½ cents. Naturally the early periods would be higher and taper off as the bonds were amortized. Converting the usage tax into the same terms it is roughly 5 cents annually per $100 assessed valuation in addition.

It is not contemplated that all of the fourteen districts be established at once, but as required. There are four of the largest districts that should be established at once, others probably five, ten or fifteen years hence.

There is only one district out of the fourteen that does not indicate that it would be more economical than the private village system, all of the others show marked savings from the outset.

The prime object of the county system is for better hygienic conditions and when there is added to this a marked economy there seems to be but one course to pursue.

It is conceded that the business of disposing of sewage is rather technical and complicated and requires the constant attention of chemists and engineers. Under the system of each village for itself there is little doubt that chemists would not participate in the operation, because the cost would be too burdensome if each village engaged its own chemist. On the other hand, we have in the county system the oppor-

tunity to set up a modern laboratory with a staff of competent chemists, who would not only be in a position to watch plant operation, but could also investigate sea water and potable water pollution. The cost of such a staff spread over the assessed valuation of the county would be insignificant to the individual taxpayer.

The Board of Supervisors, acting in the capacity of Commissioners of the Nassau County Sanitation Commission, have caused this report to be made and have presented it to the public of the County as a straightforward business-like solution of what is fast becoming a knotty problem.

It will be necessary to inform and educate the public of the desirability of solving their sanitation problem in some comprehensive manner and it is apparent that the county is the best unit of the Government through which this can be accomplished.

Because of lack of time I have not gone into detail in this address on many of the technical points. A complete report is in printed form and I have no doubt that any interested engineer may obtain a copy upon application.

Operators' Reports and Suggestions

W. D. Hatfield
219 Linden Place, Decatur, Illinois

SCHENECTADY, N. Y., SEWAGE TREATMENT WORKS

REPORT OF OPERATION, 1935

By M. M. Cohn,

Sanitary and Testing Engineer

The sewage treatment units operated continuously during the entire year and continued to perform their functions with high efficiency. The effluent discharged into the Mohawk river was well clarified and well oxidized and contributed no organic load on the stream. The plant continued to treat about two-thirds of the city's sewage flow, the balance being discharged into the river in the raw state through the existing outlets.

As has been the case in past years, the major operating difficulties were experienced in the Imhoff tanks, due to the greatly overloaded condition of these units and their scum-forming tendencies. The usual work was carried out on the bar rack, trickling filters and sludge drying beds.

1. Sewage Flow

The sewage flow treated by the plant was delivered to the trunk sewer by the sewage pumping station and by gravity. The average daily flow treated was 8.9 m.g. and the average daily pumpage was 5.6 m.g. Thus, 63 per cent of the sewage was pumped.

The treated flow varied from a monthly average of 9.4 m.g.d. during the cold month of February to a minimum of 8.4 m.g.d. in July. The high flow average came as a result of the excessive use of water to prevent faucet freezing and the low flow resulted from the dry weather experienced in May. The waste of water during winter nights should be controlled.

2. Stationary Bar Rack

A total of 197.5 cubic yards of wet trash were captured by the rack, transferred to the plowing field and disposed of by shallow plowing. This plowing was carried out weekly during the warm month. The winter screenings were stored at the dump trestle and disposed of in bulk by plowing in the spring.

Monthly trash capturings varied from a maximum of 20.0 cubic yards in December to a minimum of 14.5 cubic yards in January. The average removal for the entire year was 1.6 cubic feet per million gallons, a normal quantity for the Schenectady plant.

3. Imhoff Tanks

The nine Imhoff tanks were in continuous operation during the year, under the same solids overload conditions which they have experienced in recent years. In spite of these adverse conditions, the tanks maintained high sedimentation efficiencies and produced a satisfactory effluent and a drainable sludge.

In point of labor expended, Imhoff tank operation was not as costly as in previous years. Routine operation of the tanks is never difficult. During the sludge storage period of the winter months the tanks usually require increased attention to prevent the formation of solids in the sedimentation compartment above the slots. Again, in the spring and summer much effort is devoted to breaking up and driving down the heavy scum blanket that forms in the gas vents. During the past year this type of work was not quite as demanding as in the past.

During the winter of 1934–35, less difficulty was experienced with solids forming in the sedimentation compartment, and less time was devoted to chaining and squeegeeing the tanks. Hosing of the gas vents was reduced during the spring months in order to prevent the softening of the scum and the production of foaming in the badly deteriorated timber gas vents. During the summer, the amount of scum hosing carried out was probably less than in recent years.

The tanks maintained high sedimentation efficiency during the entire year. The removal of suspended solids varied from a month average of 76.1 per cent in June to a minimum monthly average of 64.1 per cent in November. The yearly average of efficiency of removal of suspended solids was 71.3 per cent, a creditable efficiency in view of the adverse tank conditions. The removal of settlable solids, the true index of tank efficiency, varied from a maximum monthly average of 99.0 per cent in June to a minimum monthly average of 85.3 per cent in November. During eight months, the average removal of suspended solids was over 70 per cent. During six months, the removal of settlable solids was 90 per cent or over.

It has always been shown at Schenectady that the active hosing of the gas vents for the purpose of breaking up the scum results in turbulence in the digestion compartments and passage of solids-laden liquor up through the slots into the flowing sewage. As a result, active hosing tends to reduce sedimentation efficiencies. The beneficial effect of reduced scum hosing during the spring months is shown by the high efficiencies maintained in May and June. On the other hand, scum hosing was active in November in connection with the removal of sludge to the lagoon, and tank efficiencies fell.

The tank effluent was normally free from visible suspended solids during the year. The yearly average suspended solids in the effluent amounted to 57 p.p.m. Monthly averages varied from a high of 72 p.p.m. in November to a minimum of only 46 p.p.m. in February. Auxiliary screens of ¼ in. hardware cloth mesh, located at the effluent weirs of the tanks, captured much floating matter such as garbage, matches and kitchen grease.

The tanks were kept skimmed at all times by a continuous program of hand maintenance. All skimmings were deposited in the gas vents, efforts being made to distribute the material among about one-third of the 144 vents. Most skimmings were food remnants and kitchen sleek. Digestion was satisfactory and no harmful effect was produced on the sludge.

The inlet piping system which distributes sewage throughout the tanks was kept free from clogging by a routine program of rodding and pressure blowing.

An active program of sludge drawing was initiated in April and continued through November. A total of 12,632 cu. yd. of sludge was removed from the tanks, 10,966 cu. yd. of which were drawn to the sludge drying beds and 1,666 cu. yd. were drawn to the lagoon. No lagooning of sludge was carried out until the end of the season when an attempt was made to clear the tanks for the difficult winter storage period ahead. The monthly removals of sludge from the tanks varied from a maximum of 2,264 cu. yd. in August to 618 cu. yd. in April when the sludge removals were started.

4. Sludge Drying Beds

The 10,966 cu. yd. of sludge drawn to the drying beds were of satisfactory quality and it dried readily when atmospheric conditions were favorable. The sludge was drawn to the beds to a depth of from 10 to 12 in. and dried to a cake of forkable depth. It was found that a crew of two cleaning men and one team handled the beds in satisfactory manner and were available for removing sludge from any bed approximately when the sludge was in suitable condition for removal. The moisture content averaged 95.22 per cent for the year, the organic solids content averaged 67.85 per cent, the drain, or clear water separated in 24 hours, averaged 61 per cent, the specific gravity average 1.008 and the pH averaged 6.8.

A total of 3,065 cu. yd. of air-dried sludge was produced, 2,765 cu. yd. being removed from the beds and 300 cu. yd. being on the beds awaiting removal by the local farmers. The sludge removed varied in moisture content from 65 per cent to 75 per cent depending on weather conditions. When placed in storage piles in the fields awaiting sale to farmers, the sludge was further reduced in moisture and shrinkage was marked.

Sale of sludge was more active than in recent years. Farmers removed the sludge for compositing during the entire summer. The City of Amsterdam removed 300 cu. yd. of sludge for use on its municipal golf course. At the present time, all of the field-stored sludge has been removed and the farmers are now removing sludge from the drying beds, in partially dried condition. Removal of sludge from the drying beds by the farmers has saved the City considerable funds in labor, in addition to making more bed space available for the sludge drawing program.

Continuation of the demand for sludge indicates that local farmers and householders are aware of the soil conditioning effect of the material. Many of the customers have been drawing sludge for ten years or more, indicating that the material continues to render good service. An attempt was made to interest farmers in pulverized sludge, at an increased price.

5. Dosing Tanks

The dosing tanks functioned satisfactorily during the year, with a minimum of attention. The dosing counters continued to serve as the only means of flow measurement at the plant. Recheck of calibration curves for the three tanks indicated that reasonable gaging accuracy was possible with the counter.

6. Trickling Filters

The three acres of trickling filters performed with highly satisfactory nitrification efficiencies during the entire year, and maintained a highly satisfactory physical condition. The total oxidized nitrogen contained in the final effluent averaged 9.412 p.p.m. for the entire year. The filters did not experience any sag in efficiency during any one season of the year. The uniformity of the high efficiencies is shown by the fact that the lowest monthly total oxidized nitrogen content of the effluent was 8.159 p.p.m. and the highest 9.699 p.p.m.

Table I.—*Sewage Treatment Works, Schenectady, N. Y. Operation of Treatment Units—1935*

Month	Bar Rack Screening Removal		Imhoff Tanks				Sludge Beds	Trickling Filters		
			Efficiency Removal of					Nozzles Cleaned		Stability
	Cubic Yards	Cubic Ft. Per M. G.	Suspended Solids	Suspended Alb. NH_3	Settling Solid	Sludge Drawn Cubic Yards	Dry Sludge Removed Cubic Yards	Per Day	Per M. G.	Per Cent
December	20.0	1.8	70.7	51.4	87.7	....	...	153	16	100.0
January	14.5	1.5	73.7	45.3	91.6	....	...	118	14	100.0
February	20.0	2.0	75.6	53.3	91.7	....	...	113	12	100.0
March	15.0	1.5	69.1	46.6	86.7	....	...	126	14	100.0
April	16.0	1.6	68.5	41.8	86.0	618	...	95	11	99.0
May	15.0	1.5	74.2	51.5	96.0	1416	268	114	13	98.2
June	17.0	1.7	76.1	50.0	99.0	1649	600	91	10	98.3
July	14.0	1.5	71.9	53.7	90.0	1622	530	80	10	99.6
August	16.0	1.6	70.3	50.0	89.5	2264	543	63	7	99.3
September	17.0	1.7	74.7	45.0	93.5	1776	497	65	7	99.8
October	16.0	1.5	66.3	51.4	88.2	1710	267	80	9	98.8
November	17.0	1.7	64.1	42.6	85.3	1577	360 *	109	12	98.1
Average	16.5	1.6	71.3	48.5	90.4	....	...	101	11	99.3
Total	197.5	..	....	....	....	12,632	3065	...	..	....

* 300 Yards remaining on beds.

TABLE II.—*Sewage Treatment Works, Schenectady, N. Y. Average Analyses, Sewage and Effluents—1935*

Constituent		Crude Sewage	Tank Effluent	Final Effluent
Free Ammonia	Parts Per Million	12.2	16.3	5.4
Total Albuminoid Ammonia		5.79	4.07	2.54
Suspend. Albuminoid Ammonia		3.95	2.02	1.54
Nitrites		....	....	0.629
Nitrates		....	....	8.783
Chlorides		36.0	33.4	31.3
Total Solids		575	439	391
Suspended Solids		193	57	30
Settling Solids		151	....	....
Dissolved Oxygen—% Saturation		9.6	19.2	32.6
Biochemical Oxygen Demand		113	76	12
pH Value		7.1	7.1	7.1

A highly stabilized effluent, of low oxygen demand, was consistently discharged to the river. The average 5-day B.O.D. of the effluent was 12 p.p.m. for the year, the range being from a monthly average of 16 p.p.m. to a low of 11 p.p.m. The methylene blue stability of the effluent averaged 99.3 per cent for the year. The lowest monthly stability average only fell to 98.2 per cent and four months experienced effluent stabilities of 100 per cent. Of the 281 days during which effluent samples were composited, the stability was 100 per cent on 269 days or 96 per cent of the time. The average B.O.D. reduction through the plant was 89.3 per cent for the year.

The effluent was comparatively free from suspended solid during the entire year. The average suspended solids for the entire year was 30 p.p.m. or 27 p.p.m. less than in the tank effluent. Again, as in recent years, the filters continued to discharge solids at a low, regular rate throughout the year, as evidenced by the fact that the highest monthly average suspended solids in the effluent was 36 p.p.m. and the lowest 25 p.p.m. This continuous unloading has produced the consistent efficiency of the filters.

The high filter efficiencies are even more noteworthy in the light of submergence conditions which existed during the entire year. The Schenectady filters are but five feet deep, and they are constructed at such level as to permit partial flooding of the body of the filters by fluctuations in river levels.

During the past twelve months, the filters were flooded to a level above the filter underdrains for 233 days or 63.7 per cent of the time. During 10 months, the filters were so flooded over 50 per cent of the time.

In addition to the submergence given the beds by river surges, the beds were completely inundated on 21 days during the summer season for fly control purposes. It is important that the effluent stability was normally 100 per cent a day after the beds were flooded for fly control, thus showing the ability of the beds to withstand adverse operating conditions.

The beds were in splendid physical condition during the entire year. Absolutely no signs of filter pooling were noted. It was not even necessary to pick over the stone at those distribution line ends where pooling has previously occurred.

Ice formation was heavy on the beds during the winter due to intense cold and heavy snows. The beds with poorest distribution accumulated ice on some 15 per cent of the stone area. The bed fitted with Worcester nozzles had an ice accumulation over approximately 7 per cent of the area.

Psychoda fly development and transmission was kept under control by periodic flooding. During the peak fly periods, flooding was performed once weekly and this was reduced to fortnightly during cooler weather. The beds were filled in about four hours and kept submerged for 20 hours.

Nozzle clogging averaged 101 per day for the year, or 11 per million gallons of sewage filtered.

Operation Costs

Operation costs have been maintained at a low level by continuation of the same policies of labor and purchasing which were initiated in recent years.

The total bureau budget appropriation for 1935 was $33,234.84, of which approximately $32,800 will be expended. Salary rebates have amounted to $1,300 during the year while the out-of-bureau services rendered in the form of personal services can be conservatively estimated to have cost the bureau $2,500. Thus, the net cost of sewage disposal, including pumping and treatment, was $29,000.

Of the total cost of operation, $18,600 has been expended for sewage treatment works and $10,400 for sewage pumping station operation. Unit costs are as follows:

Cost of sewage treatment	$5.72 per million gallons
Cost of sewage pumping	3.20 per million gallons

OHIO CONFERENCE ON SEWAGE TREATMENT, 1935

GADGET CONTEST

Dear Dr. Hatfield: I am enclosing photographs and descriptions of the three winners of the gadget contest at our 1935 Conference. In this connection I might say that the first prize is ten dollars, the second prize is five dollars, and the third prize is a year's subscription to the Sewage Works Journal. The awards are made on the basis of ballots cast by members of the Conference.

First Award.—The first award went to W. A. Yackley, chief chemist at the Dayton sewage treatment plant for his so-called "Flip Eliminator." This device is so simple that there was practically no description accompanying the device as it almost speaks for itself.

Second Award.—The second award went to Joseph Wein for his gadget described as an "Imhoff Tank Floating Vacuum Skimmer and Gas Vent Scum Breaker." Mr. Wein is assistant mechanic at the Akron sewage treatment plant. You will recall that T. C. Schaetzle is superintendent of this plant.

Third Award.—The third award was made to another mechanic at the Akron plant, M. D. Krunich, and is designated as "The E–Z Get It Water Level Recorder."

I neglected to add that M. W. Tatlock is superintendent of the Dayton plant where Mr. Yackley, winner of the first award, is located.

Yours truly,

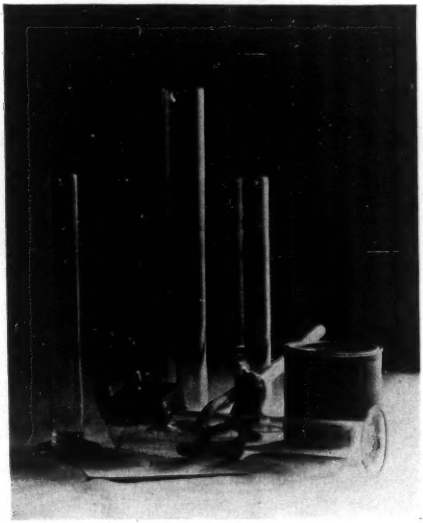
F. D. Stewart,
Principal Assistant Engineer

FIRST AWARD

FLIP ELIMINATOR

By W. A. Yackley, *Chief Chemist,*
Dayton Sewage Treatment Plant

Graduate *always* delivers the *exact volume.*
Hole can be drilled in one minute.



SECOND AWARD

IMHOFF TANK FLOATING VACUUM SKIMMER AND GAS VENT SCUM BREAKER

By Joseph Wein, *Ass't.-Mechanic,*
Akron Sewage Treatment Plant,
Botzum, Ohio

I. Instructions for Building This Type of Skimmer

Part 1: Float

The float is made like a box. The metal is cut 48 in. long and 13 in. wide. It is then formed into a tank 6 in. by 18 in. by 13 in. without ends. The ends or heads, which are separate pieces, are then soldered into position. The back end or head is placed flush, and the front end is set back 1 in. from the end to permit riveting and soldering of the scum compartment to the box proper. The handle frame (lawn mower type) is made of strap iron ¼ in. by 1 in. and is bolted to the tank 3½ in. from the back and 1 in. from the top

of the float. It extends forward to a guide 10½ in. from the end of the float made of strap iron ⅛ in. by ¾ in. by 3 in.

Part 2: Scum Compartment

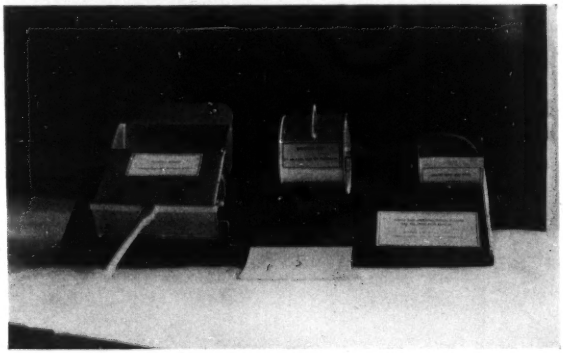
The scum compartment is built on a 45° angle, being 3½ in. wide on the bottom with a 2 in. suction line in the center, and a 7 in. opening on top. A 2 in. skimming lip extends to the front. This section is fitted inside the 1 in. extension on the float, riveted and soldered. Number 18 gauge galvanized sheet metal is used throughout.

II. Operation

This skimmer as designed has a dual purpose. It cleans the surface of the Imhoff Tank flow chamber and breaks the hardened crust or settles the foaming scum in the gas vents.

The suction for this skimmer is obtained through a small portable "Homelite" pump, which also produces pressure enough on the discharge end to break the crust or settle the foam in the gas vents.

This skimmer is a floating type. By raising the handle which is mounted on the rear of the float the skimming lip is lowered under the surface of the water and the scum flows into a compartment in front of the float where the suction on the bottom draws it into the pump and discharges it into the gas vents.



THIRD AWARD

THE E-Z GET IT WATER LEVEL RECORDER

By M. D. Krunich, *Master Mechanic,*
Akron Sewage Treatment Plant,
Botzum, Ohio

I. Purpose: To obtain accurate elevations of water or sewage in Piezometer tubes.

II. Construction:

1. Hollow recording rod.

(*a*) Head: The head of this rod is made of brass of such a size as to hold a "Pencil" flashlight bulb. It is threaded on the inside for the bulb and on the outside with SAE threads to fit the battery case (*b*).

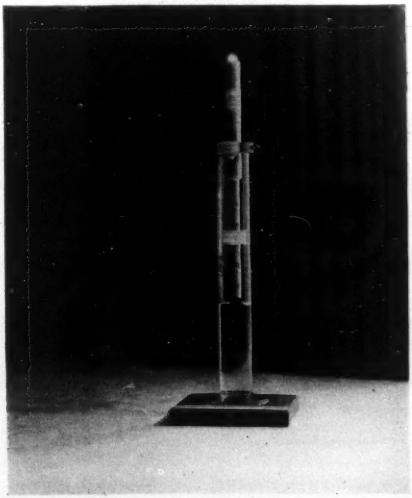
(*b*) Hollow battery case: This is a hollow rod 5⅝ in. long, ¾ in. outside diameter and ½ in. inside diameter. This length is sufficient regardless of the length of the graduated float rod (*c*), as it is long enough to hold 2 size AA batteries and to accommodate the inside threads for the head at the top and the graduated rod at the bottom.

(*c*) Graduated float rod: This is a hollow rod ½ in. outside diameter and of whatever length may be required for the problem at hand. The top or end which fits into the bottom of the battery case is threaded with SAE male threads 1⅜ in. long. At the bottom of the graduated float rod the SAE female threads are 11/16 in. long. Both ends are plugged with a filler, with an opening through the filler just large enough for the 3/32 in. contact rod to pass through freely. The rod can be graduated into any units depending upon the accuracy desired.

(*d*) Cork assembly,—Cork, plug, spring and contact rod: This consists of a cork approximately 1⅛ in. in diameter and ¾ in. high through which the 3/32 in. contact rod passes. The contact rod is fastened firmly at the top and bottom of the cork with a small soldered washer. This rod is of sufficient length to just pass through the filter at the top of the graduated rod. It is brought to a sharp point for contact with the batteries. At a point 1½ in. above the top of the cork a collar is soldered onto the contact rod. Between this collar and the top of the cork is placed the threaded bottom filler plug which fits into the graduated rod (*c*). Between the top of the filler plug and the collar on the contact rod is a small coil spring made of No. 2 to No. 4 piano wire. The collar and plug are so spaced as to put the spring under slight compression. The compression is just sufficient to keep the contact point away from the batteries and to allow a space of about ⅛ in. between the bottom of the graduated rod and top of the cork.

2. Movable collar.

This may be made of any available material and any desired thickness and outside diameter. The inside diameter, however, must be just an easy fit on the graduated rod as the collar is used merely to fix permanently the gauge reading in the same manner as a vernier is used on a surveyor's rod. The thumb screw is used to hold the collar in place for reading.



3. The graduated cylinder.

This represents the Piezometer tube and is selected of approximately the same inside diameter as the Piezometer tube so as to determine the water displacement by the cork in the Piezometer tube.

III. Operation:

Having determined the proper overall length of the graduated float rod for the condition at hand, the parts *a* to *d* are assembled and tested in a glass cylinder of the same diameter as the Piezometer tube. By placing the entire unit in the glass cylinder partially filled with water, the cork will be submerged somewhat. The depth to which the cork is submerged to make contact with the batteries so the bulb will light gives the zero point from which to graduate the rod.

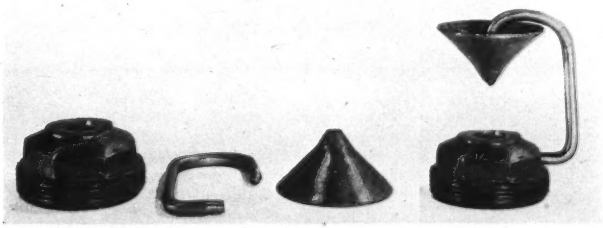
After the rod is graduated the E–Z Get It is ready for use. Now submerge the instrument in the Piezometer tube. When the bulb lights drop the movable collar to the top of the Piezometer tube, turn the thumb screw so as to hold it in place. Remove the entire instrument and read the rod at the under side of the collar. Knowing the elevation of the top of the Piezometer tube and the rod reading, the water or sewage elevation can be obtained direct. The degree of accuracy depends upon the refinement in calibrating the graduated float rod.

A "SCOTCH" SPRINKLING FILTER NOZZLE

By W. D. Hatfield

The three acres of sprinkling filters at Decatur, Illinois, were equipped with Taylor type nozzles having an orifice opening of ¾ in. diameter. Half of the spindles were of the screw type and half were of the lock type. Due to the corrosive action of the sewage the spindles (particularly the lock type) became so loose that they would blow out or fall over to one side, thus greatly reducing the efficiency of distribution.

Many of the old nozzles have been rejuvenated as shown in the accompanying photograph at a material cost of less than 3 cents per



nozzle. The repairs were made by our regular labor, the night shift operator cutting and threading the brass rods and sawing off the spindle of the distributor. The day operator drilled and tapped the ¼ in. holes in the distributor and the base of the nozzle. Our foreman bent the ¼ in. brass rods around a die which he made and assembled the

nozzles as shown in the photograph. The ¾ in. orifice was bushed down to reduce the orifice proportionately for the area of the spindle.

We have had about 500 of these remodeled nozzles in operation since November, 1935, and have found them very satisfactory. The nozzles do not clog easily because of the unobstructed hole and because of the fact that the cross member which originally supported the spindle has been removed. We feel that we have a new nozzle for 3 cents which is equal to or better than the original.

Now, come on the rest of you operators!! I want a little "Scotch" contest. Can you beat it?

Editorial

SEWAGE TREATMENT IN OUR LARGE CITIES

Remarkable achievements have been accomplished during the past year or two in the construction or completion of sewage treatment works for the largest cities of America. This rapid progress has been possible mainly because of the P.W.A. loans and grants, without which many projects would have been greatly delayed or abandoned.

A brief review of plants completed, under construction or ready for immediate construction, with reference to size, type, and unique features, may be of interest. No statistical data have been compiled, although data on population, costs, and degree of treatment might warrant thorough study by a committee of our Federation.

The largest sewage treatment plant under construction in America is the Southwest Side Works of the Sanitary District of Chicago. This is an activated sludge plant, with a capacity of 400 m.g.d. which may be extended to 600 m.g.d. by introducing sewage from the West Side Works, if found possible. This Southwest Side plant will go into operation in 1937, and will probably for many years hold the distinction of being the largest activated sludge plant in the world. It will also be noteworthy because of the absence of digestion tanks, and the use of mechanical filtration and incineration of sludge, in conjunction with a steam power plant. The unit costs of construction and operation promise to be exceptionally low.

New York's activated sludge plant at Ward's Island should also be in operation in 1937. This plant, with a capacity of 200 m.g.d., will result in great improvement of sanitary conditions in the East and Harlem Rivers. In addition to this plant, New York now has the Coney Island chemical precipitation plant of 30 m.g.d. capacity, and several other plants of similar size or larger are being designed or constructed.

The Washington sedimentation plant is nearing completion below the nation's capital, designed to relieve the pollution of the Potomac River. This plant is noteworthy for the pioneer installation, on a large scale, of elutriation of digested sludge prior to partial dewatering on vacuum filters. It is proposed to remove the sludge cake, with 70 to 75 per cent water, and to dispose of this material without further dewatering.

The Easterly Works at Cleveland, an activated sludge plant, will also soon be ready for operation. The feature of this plant is the pumping of sludge to the Southerly Works for digestion and disposal. Cleveland is now working on the design of the Westerly Works, which with the Southerly Works, a trickling filter plant, will complete Cleveland's sewage treatment program.

The Columbus activated sludge plant is second largest in Ohio. This project is noteworthy for the use of storm water tanks along the intercepting sewer. These tanks go into service during storms and the settled solids are flushed back into the sewer and carried to the treatment works in dry weather.

Buffalo and Detroit are preparing to construct sedimentation plants, and chlorination of the effluents will be an important factor in both. In both, also, bacteriological efficiency is paramount, to protect downstream water supplies. Very great dilution of effluent (in the Niagara and Detroit Rivers) obviates any necessity for complete treatment of sewage. The Buffalo problem is complicated by the present pollution of the Buffalo River by industrial wastes, which are of quite varied nature. Industrial wastes are probably of less importance at Detroit.

Minneapolis-St. Paul is the center of attention at present because of the decision to use rectangular settling tanks 290 ft. long, and to provide for chemical treatment at periods of low flow in the Mississippi River. The Minneapolis-St. Paul project is of outstanding interest because of the adoption of sedimentation plus chemical treatment in place of activated sludge treatment for disposal in rather limited volumes of dilution water. A disturbing factor at present is the lack of any provision for treatment of the

packinghouse waste at South St. Paul, a rather large factor in pollution of the Upper Mississippi River.

Somewhat similar to the Minneapolis-St. Paul plant is the chemical precipitation plant now ready for construction at Denver, with the exception that the Denver plant will use chemicals for a substantial portion of each year, whereas at Minneapolis sedimentation alone may be sufficient most of the time. Denver is noteworthy in having adopted the use of magnetite filters for final clarification of the effluent, and bids on such filters have also been taken at Minneapolis.

Milwaukee and Indianapolis have greatly enlarged their activated sludge plants, the former with a change from ridge-and-furrow to spiral-flow type of air diffusion.

Jumping to the West Coast, we find Los Angeles County (Messrs. Warren and Rawn) proceeding with the construction of a tunnel for disposal of a settled effluent at sea. The City of Los Angeles still operates only a screening plant at Hyperion.

San Francisco is starting construction of a sedimentation plant in Golden Gate Park. This plant is of interest, although small, because of its critical location with respect to odors and appearance.

Portland and Seattle have apparently made no substantial progress toward sewage treatment, because of plentiful dilution of raw sewage. However, reports and projects involving sedimentation have been considered.

Passing to the large cities of Canada, we find Winnipeg engaged in construction of sewage treatment works. It is reported that a large number of comminutors are proposed for use in this project. Toronto has considered a comprehensive scheme of sewage collection and treatment for quite a number of years, but as yet nothing has been done. The North Toronto activated sludge plant, however, has recently been enlarged. Montreal is considering sedimentation works and some activity is apparent at Quebec.

Returning to the United States, it is now easy to refer to the large cities, because there are so few, where very little if any progress has been accomplished toward sewage treatment. St. Louis probably leads the list, because at St. Louis pollution of the Mississippi River has deliberately been increased by grinding of garbage and dumping the putrefactive mass into the river. The Father of Waters can probably take it, but it is strange that the spirit of St. Louis, so alert and commendable in other civic undertakings, has been so sluggish in its consideration of sewage treatment. Kansas City, also a city of strong civic conscience, has made no provision for sewage treatment.

Louisville, Cincinnati and Pittsburgh, on the Ohio River, have no sewage treatment. These three cities are largely responsible for the highly polluted condition of the Ohio River. All possible practicable measures have been taken along the Ohio River to extend water treatment to its limit of effectiveness, but very little progress has been made in installation of sewage treatment plants. How much longer must the water treatment plants handle this load?

In summarizing this brief review, it is apparent that a great impetus has been given to the treatment of sewage in the United States during the past few years, and nearly all large cities have felt the urge and need to provide some sort of sewage treatment. The decisions as to types and degree of treatment have not been made hastily, but competent engineering advice has been freely sought and sound decisions of the engineers have been followed. The decade 1930–40 will undoubtedly be recorded as the one of greatest progress in the development of sewage treatment in America.

F. W. M.

Proceedings of Local Associations

PENNSYLVANIA SEWAGE WORKS ASSOCIATION

Tenth Annual Conference, State College, Penna., June 24, 25, 26, 1936

The Tenth Annual Conference of the Pennsylvania Sewage Works Association was held at the Pennsylvania State College, State College, Penna., on June 24, 25 and 26, 1936. A total of 131 persons registered for various sessions.

The annual Golf Tournament, which is held in co-operation with the Pennsylvania Water Works Operator's Association, took place Wednesday afternoon, June 24th, under the direction of E. M. Jones of the Simplex Valve and Meter Company. Mr. Jones was chairman of the Golf Committee, which consisted of representatives of manufacturers and technical journals. This committee collected 14 unusually valuable prizes, which were awarded the winners at the Sewage Works Dinner which was held Thursday evening, June 25th.

The technical sessions were opened Wednesday evening by J. R. Hoffert of the Pennsylvania State Department of Health, who gave an interesting paper concerning the utilization of reports and diagrams by the sewage treatment plant operator.

On Thursday morning the meeting was opened with an address of welcome by J. O. Keller, Head of the Engineering Extension, Department of the Pennsylvania State College.

The first paper of Thursday's technical session was entitled "Aero Filtration of Sewage and Industrial Wastes" by Professor H. O. Halverson of the University of Minnesota, who described tests run on a trickling filter in which rates as high as 26 m.g.d. per acre were maintained without rest periods. The discussion of Professor Halverson's paper was opened by I. O. Lacy, Sanitary Engineer, U. S. Resettlement Administration, Washington, D. C., who stressed the need for further experimental work to determine whether such high rates of filtration can produce efficient results over long periods.

Henry W. Taylor, Consulting Engineer of New York City, described the "Operation of the Canajoharie and Herkimer, New York, Waste Disposal Plants" where both municipal wastes and sewage are disposed of in single plants. A written discussion of this paper, prepared by C. C. Agar, Assistant Engineer, New York State Health Department, was read by L. D. Matter, Secretary of the Pennsylvania Sewage Works Association.

The early part of Thursday afternoon was devoted to a "Sewage Works Clinic" conducted by H. M. Freeburn of the Pennsylvania State Department of Health. This clinic was conducted in a question and answer manner and covered the following subjects, the questions being

answered by the men whose names appear opposite the subjects in question:

(*a*) "Design and Cost"	F. S. Friel, Consulting Engineer, Albright & Friel, Inc., Philadelphia, Penna.
(*b*) "Operating Fundamentals"	L. V. Carpenter, Assistant Professor Hydraulic and Sanitary Engineering, New York University, New York City.
(*c*) "Imhoff-Trickling Filter Plant Operation"	R. R. Cleland, Sanitary Engineer, Dept. of Grounds and Buildings, Pennsylvania State College, State College, Penna.
(*d*) "Operation of the Separate Sludge Digestion Plant"	C. H. Young, District Engineer, Pennsylvania State Department of Health, Meadville, Penna.
(*e*) "Activated Sludge Plant Operation"	A. J. Fischer, Sanitary Development Engineer, The Dorr Company, Inc., New York City.
(*f*) "Hints for Chlorinator Operators"	A. E. Griffin, Chemist, Wallace & Tiernan Company, Inc., Newark, N. J.

Following the Sewage Works Clinic two interesting papers were presented which dealt with the construction of sewers and sewage treatment works under W.P.A. sponsorship. Paul A. Keiser, Pottstown, Pa., described methods used in constructing an activated sludge plant at Ambler, Penna., with W.P.A. funds while Robert H. Craig, Consulting Engineer, Harrisburg, gave a very clear description of projects conducted under W.P.A. and P.W.A. control and offered some very excellent advice as to the supervision of such projects by the municipalities for which the work was being done.

The Annual Dinner was held at the Nittany Lion Hotel, Thursday evening, under the capable leadership of H. E. Moses, Assistant Chief Engineer of the Pennsylvania State Department of Health. This dinner was kept alive by Mr. Moses' humorous stories and introductions, special selections by the Activated Sludge Quartet and by group singing under the able leadership of Dave Evans of Reading.

The principal speaker of the evening was W. L. Stevenson, Chief Engineer of the Pennsylvania State Department of Health, Harrisburg, Penna., who spoke on "Development and Progress in Sewage Treatment."

Following Mr. Stevenson's address, Toastmaster Moses, on behalf of the Association, presented to the Pennsylvania State College an oil painting which portrayed the development of sewage disposal during the period 1776 to 1936, inclusive. Professor Elton D. Walker accepted the Association's gift on behalf of the College with appropriate remarks.

The Friday morning session was devoted to a plant operation symposium. The first paper by Harry J. Krum, City Chemist of Allentown, described the operation of the Allentown Sewage Treatment Plant (Imhoff-Trickling Filter) at which considerable trouble has been experienced due to the formation of scum and foaming in the Imhoff tanks. Discussion on this paper was opened by E. Sherman Chase, Metcalf & Eddy, Consulting Engineers, Boston, who designed the plant.

The second operating paper was presented by George P. Searight, Borough Manager, Carlisle, who described the new Carlisle Sewage Treatment Plant (Sedimentation-Rotary Trickling Filter-Separate Sludge Digestion). At this plant the rotary distributors on the trickling filter are operating under a head of only thirteen inches. The designer of the plant, Professor E. D. Walker of Pennsylvania State College, opened the discussion of this paper.

The final operating paper was given by E. C. Miller, Plant Superintendent of the Dearborn, Michigan, Sewage Treatment Works (Chemical Precipitation—Magnetite Filter). Mr. Miller described in detail the methods used at Dearborn in handling sludge on vacuum filters followed by incineration. W. H. Baumgartner, Sanitary Engineer, Grasselli Chemical Company of Cleveland, opened the discussion of Mr. Miller's paper.

At the business session of the Association the following officers were elected:

President	R. R. Cleland
First Vice President	F. S. Friel
Second Vice President	C. F. Grace
Secretary-Treasurer	L. D. Matter
Editor	J. R. Hoffert
Representatives on Board of Control	(C. A. Emerson, Jr., H. E. Moses)

Inasmuch as a Sewer Rental Law is now in force in Pennsylvania, the Standing Committee in the Association on Sewer Rentals was discharged with thanks for its work.

Because of such relatively low attendance at the short course school for sewage treatment plant operators conducted by Pennsylvania State College in 1935 and 1936, it was decided that this instruction should be carried out along different lines. Accordingly, a committee was appointed to look into the matter of conducting a correspondence course in co-operation with Pennsylvania State College under which plan it would be possible to give instruction to the various grades of operators,

which work might be supplemented with special work at the College at the end of the correspondence course. It was pointed out that there is no suitable text for such correspondence courses and the committee was authorized to investigate the possibility of having such a text written by capable operators.

The Committee on Operator Licensing was able to present its proposed licensing bill to the members only a few days before the annual meeting, therefore it was decided to defer action upon this proposed bill until a later date in order to give the members an opportunity to study the same.

The Association also authorized the changing of the Constitution to provide for an increase of annual dues of members to $2.50. The purpose of this increase is to conform to the proposed increases in subscription rate to *Sewage Works Journal,* which it is expected will take place as of January 1, 1937.

The Association also took notice of the excellent work of F. S. Friel, Consulting Engineer of Philadelphia, as chairman of the Program Committee and passed a resolution of thanks to Mr. Friel.

The Executive Committee was authorized to pay the Secretary of the Association a small salary.

The 1937 Annual Meeting will be held at State College, June 21, 22, and 23.

L. D. Matter, *Secretary-Treasurer.*

Reviews and Abstracts

H. W. STREETER

SEWAGE DISPOSAL AT EXETER

BY W. H. HOYLE

Paper presented at the Summer Conference of the Institute of Sewage Purification at Exeter, July 2, 1936, and reprinted—slightly condensed—in *The Surveyor*, 90, 7–9 (July 3, 1936).

The original septic tanks were installed at Exeter by Cameron and served satisfactorily for many years until they were overloaded. Three separate sewage works were in use, two consisting of septic tanks and contact beds and a small plain sedimentation and land irrigation plant. These three plants have now been replaced by a single plant on a new site of 43½ acres lying between the River Exe and the Exeter Ship Canal.

The sewage—2.5 m.g.d.—is conducted to the plant through a main of 48 inch and 54 inch pre-cast concrete tubes and under the river through two lines of 27 inch cast iron pipe encased in concrete.

The new plant is designed to handle three to six times the dry weather flow, the excess to be given preliminary treatment. The principal elements are:

Detritus Tanks and Screens.—Three detritus tanks, each of 25,000 gallons capacity, are equipped at their inlet ends with coarse bar screens with 3 inch clearance cleaned by hand rakes. The tanks are used in rotation, two at a time, and are cleaned by an electrically operated grab mounted on a traveling crane. Grit is hauled to a tip and the lighter matter pumped to the primary sewage digestion tanks.

Screen House.—"Simplex" grab type mechanically-cleaned fine screens with ¾-inch clearance between bars receive the effluent from the detritus tanks. These screenings are transported by a band conveyor and buried.

Preliminary Tanks.—Sixteen Dortmund type settling tanks have a total capacity of 833,000 gallons. The sludge is discharged hydrostatically to a sludge well and from thence is pumped to the digestion tanks.

Aeration Tanks.—Eight aeration tanks, with a total capacity of 1,682,000 gallons or 16 hours d.w.f., are each equipped with five Simplex Surface Aeration cones. An adjustable weir on each tank outlet provides a method for varying the submergence of the cones, thereby changing the aeration as required.

Final Settlement Tanks.—Eight Dortmund type tanks receive the aerated effluent. Their total capacity is 648,000 gallons or 6.22 hours detention period of the d.w.f. The effluent is discharged to the river and the settled sludge is pumped either to the inlets of the aeration tanks or to the works inlet as surplus sludge.

Sludge Digestion Tanks.—Two primary digestion tanks have a total capacity of 200,000 cu. ft. with a working depth of 10 ft.; three secondary units have a capacity of 189,000 cu. ft. and a working depth of 7.5 ft. A special arrangement of inlet and delivering piping provides considerable flexibility for filling, mixing, decanting and emptying of the various units.

Drying Beds.—The 64 drying beds are each 50 x 50 ft. in plan, are all underdrained and composed of graded gravel with a top sand layer.

OPERATION

From 8 to 10 per cent of activated sludge is carried in the aeration tanks equivalent to from 540 to 920 p.p.m. The return sludge has a water content of 99.3 per cent and varies from 1.25 to 1.5 m.g.d. of which about 80,000 gallons is surplus. Analytical results show the effluent to contain 8 p.p.m. of suspended solids and a percentage of

purification of 82.9 as measured by the "oxygen absorbed in 4 hours" test. "Bulking" has never been experienced.

About 21,500 g.p.d. of sludge is pumped to the primary digestion tanks in rotation. Sludge from the secondary tanks is pumped into the main along with the raw sludge for seeding purposes, about 25 per cent by volume being used. The separated water, about 10,000 g.p.d., was originally returned to the inlet of the preliminary settling tanks, but because this procedure caused fermentation and scum formation with carry over of solids in the effluent, this liquor is now returned to only one of the preliminary tanks.

Digested sludge is run on the drying beds in a 9-inch layer and in 15 to 20 days dries to 2 inches. As soon as a first application cracks through to the sand layer, a second and even a third dose can be applied without increasing the difficulty of removing the dried material.

To control plant odors resulting from septic sewage in warm weather, chlorine is applied at one of the sewage pumping stations at the rate of from 1.5 to 2.0 p.p.m. which has thus far been entirely effective.

J. K. HOSKINS

THE DISCHARGE OF SEWAGE INTO TIDAL WATERS

BY WILLIAM A. MCCARTNEY

Paper read before the Sanitary Inspectors' Association of Scotland and abstracted in *The Surveyor*, **89**, 703 (May 22, 1936).

The author quotes the thesis of Professor Ellis to the effect that "If crude sewage can be diluted sufficiently at small expense, its dilution is the obvious method for its disposal, provided the project is feasible and has no objections on other grounds." From this point he discusses the extent of dilution necessary and how such dilution may be most readily provided for in outfall construction.

The degree of dilution of sewage obtainable between the time it leaves the outfall and the time it reaches the surface of the sea is an important consideration. Some schemes discharge the sewage in such a depth of water and in such a way that the initial surface dilution is 1 in 30. This is accomplished in Cape Town by having four 6-inch outlets on each side of the sea end of the outfall discharging horizontally and at a considerable distance apart.

Proper location of outfalls is most important. Surface floats are helpful in the determination of currents because it is the surface floating materials in sewage which cause the trouble.

In recent years the use of steel pipes has become general for sea outfalls, because of their advantages of weight, strength and ease of laying.

J. K. HOSKINS

NOTES ON THE MANUFACTURE OF FERRIC SALTS FOR TREATMENT OF LONDON SEWAGE

BY W. P. WARLOW

The Surveyor, **89**, 771–73 (June 5, 1936)

Large scale experiments are in progress for treating London sewage at the outfalls with ferric salts (chlorinated copperas to us). Because this chemical is not readily available in the desired quantities, it has been found advisable to prepare it at the site, either by mixing solutions of ferrous sulphate and chlorine together or by introducing chlorine gas into a ferrous sulphate solution.

Details of the mixing chambers, methods of mixing and general notes on the handling of chlorine, chlorinated copperas solutions, piping, pumps, etc., are given. The precipi-

tant is added to the outfall sewage and mixed with it by means of compressed air blown through perforated pipes laid in the sewer inverts just below the point of application of the chemical. The volume of solution is held constant and the strength varied in accordance with the rate of sewage flow.

J. K. HOSKINS

SEWAGE DISPOSAL AT WAKEFIELD. REMODELLING AND EXTENSIONS OF THE CALDER VALE DISPOSAL WORKS: BIO-AERATION TREATMENT

ANON.

The Surveyor, **89**, 639–40, 680 (May 8, 15, 1936)

Wakefield, with a population of 54,330, has recently completed extensions of its sewage treatment works to handle the 2.5 m.g.d. of dry weather flow, about 50 per cent of which is contributed by industry.

The earliest type of treatment employed at Wakefield was chemical precipitation and land irrigation to which were added later bacteria beds of 2 m.g.d. capacity. These were greatly damaged by settlement of the coal workings over which they were located but the beds have now been remodelled to provide 12,900 sq. yds. of percolating filters.

Following experimental studies by the late John Haworth and L. Ives, the City Engineer, the bio-aeration method of treatment was decided upon and, with some alterations occasioned by ground settlement of as much as 5 feet, temporary construction was finished in 1926. The recently completed plant has a capacity of 3 m.g.d., sufficient to serve a population of 65,200 persons and treat 4.6 volumes of the dry weather flow. No storm water tanks are provided owing to the ability of the bio-aeration tanks to deal with large quantities of weak sewage.

Screens.—Two screen chambers contain bar screens 11 ft. high by 8 ft. wide with clear spaces of ¾-inch. They are cleaned by motor driven rakes.

Detritus Tanks.—The tanks have a capacity of 1/76 the dry weather flow, which capacity can be increased for storm flows. Detritus is removed by hand.

Settlement and Bio-aeration.—Eight preliminary tanks provide a sedimentation period of 16.8 hours after which the effluent passes through fine screens to the bio-aeration tanks. One of these is 254 by 79 ft. in plan, divided into 14 longitudinal channels. Three others, of similar construction, are 206 by 110.7 ft. in plan and divided into 18 channels affording a length of travel of sewage of 3831 ft. The total capacity of the four tanks at the normal working depth of 3 ft. 8 in. is 2.017 m.g. and the dilution period 19.4 hours. Agitation and aeration are effected by 68 paddle wheels of framed steel each 10 ft. in diameter and which make 15 r.p.m.

About 5 to 8 per cent of activated sludge is maintained in the aeration units. After passing through the aeration channels, the sewage sludge mixture passes to 18 Dortmund final settling tanks each 24 ft. square with pyramidal bottoms and providing a total capacity of 594,000 gallons or 5.7 hours detention, d.w.f. The effluent is pumped to bacteria beds and all volumes in excess of 2.3 times the d.w.f. pass to the river over an adjustable metal weir on the effluent pump suction well. Return sludge volume is measured through a Venturi meter.

Sludge Digestion.—Surplus activated sludge is pumped to open digestion lagoons providing a capacity of 5.45 cu. ft. per capita and there mixed with the sludge from the sedimentation and humus tanks. The digested sludge is run off to 3¼ acres of drying beds.

J. K. HOSKINS

DISCHARGE OF CRUDE SEWAGE INTO TIDAL WATERS

By S. B. Thomas

Paper presented before the joint meeting of the Southern and South-Western Districts of the Institution of Municipal and County Engineers, Bristol, England, June 5, 1936, and abstracted in *The Surveyor*, 89, 797–98 (June 12, 1936).

Experiments on the chlorination of crude sewage, with the purpose of improving the condition of the river Avon at Bristol, are described in this paper.

Bristol has been plagued with odor and nuisance troubles from discharge of sewage into the Avon for over 100 years, increasing population rendering various improvements ineffective from time to time. At present there are nine principal outfalls discharging to the river, upon all of which chlorination experiments have been inaugurated.

Although the complete solution of the problem involves the discharge of the crude sewage into the tidal waters of the river Severn where sufficient water is available to effect the necessary dilution, this involves a construction cost of £800,000 and annual pumping charges of £3,000. Accordingly it has been suggested that relief might be obtained by chlorination of the sewage sufficiently to render it innocuous until the dilution afforded by the Avon and the Severn could complete the purification aerobically. To determine the feasibility of this procedure, chlorination experiments were started on seven principal sewers which are subject to tide locking twice a day.

Chlorine gas was applied through W. & T. vacuum solution-feed type chlorinators to the sewage through diffusers on the sewer inverts at a rate equivalent to 25 or 50 per cent of the aggregate chlorine demand, and generally from 8:00 A.M. to 11:00 P.M. daily. River examinations indicated that the degree of oxygen saturation and the volume of water present would require ordinarily chlorination to the extent of 25 per cent of the chlorine demand to prevent odor nuisance, and that during very hot and humid summer temperatures, 50 per cent of the chlorine demand would need to be supplied. Chlorination on this basis was continued from May to the end of September. With the hot weather at the end of July odor nuisance began to be noticeable, especially during evenings at low water when the tide was on the turn and at times of high temperature and humidity. These odors were increased materially by ship propellers stirring up the river mud. The increase of chlorine to 50 per cent of the demand reduced the septicity figure of the crude sewage but the lack of capacity of the chlorination plants prevented definite conclusions being drawn concerning the improvement in river conditions. However, the mud flats, which generally became strongly septic during the hot weather, were rendered non-septic and remained so during the hot weather of August.

The conclusions drawn from these experiments are as follows:

"(1) That chlorination on a basis of 25 per cent of the aggregate chlorine demand had effectually checked the tendency of the sewage to turn septic; the three-day incubation results showing rather more favorable septicity figures than the seven-day test;

"(2) That chlorination on this 25 per cent basis had a definite but incomplete effect on odor nuisance;

"(3) The more intense chlorination had brought about a greater reduction of septicity on the crude sewage samples, but it was not possible in the first year's experiments to apply sufficient chlorine as would ensure the complete control thereby of odor nuisance on the river."

It is proposed to continue the experiments for another 12-month period on an extended basis. The author stresses the point that the unusually abnormal tide range at Bristol, resulting in movements of very large volumes of water twice daily up and down the river, renders possible the successful use of chlorination as a prevention of nuisance.

J. K. Hoskins

DESIGN AND OPERATION OF SEWAGE DISPOSAL WORKS IN SOUTH AFRICA

By E. J. Hamlin

The Surveyor, **89**, 561–62 (April 17, 1936)

The author discusses some of the differences in conditions of the two countries which render unsuitable the adoption of European standards of design for sewage treatment works in South Africa. Generally, building lots are larger than in European towns which results in lower population per acre, longer sewers per head of population, higher cost per capita and less sewage per unit length of sewer.

Rainfall.—Rainfall varies markedly over South Africa from 2 to 100 inches per annum, and intensities have exceeded 10 inches per hour and 5 inches in 25 minutes. Moreover, all the rainfall takes place within six months of the year. Every endeavor is, therefore, made to exclude storm water from the sewerage system to avoid slow velocities for dry weather flow.

Temperature.—Temperatures may vary as much as 50° F. in 24 hours. Such changes affect seriously the activated sludge method of treatment at high altitude and the behavior of sedimentation tanks especially because of the carbohydrate diet of the people.

Water Supply and Sewerage.—Water is expensive because of the stringency of the irrigation laws and the high compensations payable to riparian owners. It costs from 5d. to 7s. 6d. per 1000 gallons in different areas. Of the 164 South African towns, 33 have a population over 10,000, of which number 14 have established sewerage systems, and some method of disposal, but 7 others have works in course of construction.

Sewerage Systems.—In most towns in South Africa storm water treatment is not necessary. Separate systems are advisable and the peak flow may be assumed about twice the average rate, except where the contributory population is near the disposal works when it is usual to take the peak rate three times the average. Natural infiltration is usually very small, the figure used by the author being 9,000 gallons per minute per mile of sewer.

Screens and Grit Chambers.—Hand operated screens, usually with not less than 1 inch openings, set at an angle of 30° with the flow, are recommended and are more economical than mechanically operated ones. Grit chambers, except in hilly districts, or where night soil pails are emptied into the sewer, are rarely advisable because of the quantity of ash used under these conditions.

Sedimentation Tanks.—Plants are rarely larger than 1 m.g.d. capacity for which condition the upward flow tank is most suitable. Great variations in temperature seriously affect sedimentation where much colloidal matter or activated sludge is dealt with. Increase of the retention period or admission of septic sewage intensifies this difficulty so that the design of works must be such as to deal with these factors.

Bacteria Beds.—Because of their adaptability to varying conditions are invaluable for South African conditions. Minimum size of 1 inch for filtering material is strongly recommended.

Sludge Digestion.—Mr. F. Walton Jameson, late City Engineer of Pretoria, was a pioneer in the development of separate sludge digestion. Sludge digestion and drying offer no particular difficulties because of favorable climate. A much lower capacity per capita of separate digestion tanks is necessary than is required in Europe or America.

Activated Sludge Method.—This process can only be recommended under special conditions and where land is cheap it should probably never be employed. Its chief applications in South Africa are as a preaeration or partial treatment process. From research work conducted at Johannesburg, it is concluded that a tremendous amount of additional work must be done before the process can be recommended for the majority of future installations.

General.—The author stresses the need of trained management of sewage disposal works and suggests that the consulting engineer should be retained and held responsible for the efficient management of the works he has designed for a period of at least two years. Also, comprehensive information should be co-ordinated and made available to

simplify the task of designers, and to develop works that take into consideration the local factors existing in South Africa such as climatic conditions, action of trade wastes and the high carbohydrate diet of the population.

J. K. HOSKINS

SULPHITE WASTE LIQUOR. LABORATORY STUDY OF ITS ANAEROBIC DECOMPOSITION WHEN DISCHARGED INTO WATER BODIES

BY H. K. BENSON AND A. M. PARTANSKY

Industrial and Engineering Chemistry, **28**, 738 (1936)

Sulphite waste liquor, from the acid sulphite process for the production of cellulose from wood, contains from 10 to 12 per cent organic matter. The high oxygen demand of such a liquid is most objectionable when the liquid is discharged into a stream. A study was made to determine how rapidly and to what extent sulphite waste liquor is decomposed when discharged into a water course.

The end products of the decomposition of sulphite waste liquor by bacteria differ, depending on whether or not free oxygen is present. Under aerobic conditions, the organic matter is converted ultimately to carbon dioxide and water. Under anaerobic conditions, carbon dioxide and methane are produced. The amount of gas produced serves as a measure of the material decomposed. Also, the ratio of carbon dioxide to methane indicates whether or not intermediate products are accumulating. Under anaerobic conditions, the average extent of gasification is about 25 per cent of the carbon input. Methane production was not speeded up by altering the seed or maintaining optimum conditions, and requires a 2 to 3 month period for its completion. The lignin fraction of the liquor is decomposed much more slowly than the sugars, but its oxygen demand is only about one-fifth that of the original liquor. By fermenting sulphite waste liquor anaerobically prior to its discharge into a stream, the B.O.D. of the liquor is reduced to about one-fifth of its original value. Under proper conditions, certain organisms can convert the sulphite waste sugars into organic acids (butyric and acetic) in the course of 2 days. This procedure would not reduce the B.O.D. of the liquor materially but the acids could be removed by distillation and the residual liquor would contain only the relatively stable lignin.

P. D. MCNAMEE

SEWAGE FILTER FLY NUISANCES

BY J. P. TODD

Paper presented at meeting of the North-Western Branch of the Institute of Sewage Purification on March 6, 1936 and abstracted, with discussion, in *The Surveyor*, **89**, 449, 457 (March 20, 1936).

Of the Psychodidae family of flies, two types, *Psychoda severini* and *Psychoda alternata*, are found in sewage filters. The eggs are usually hatched within 10–14 days but at laboratory temperatures, in 6–7 days. The flies are most numerous between May and September and aerial nuisance occurs only when the mean daily temperature is above 50° F. Rain and wind prevent emergence of the fly from its breeding place.

Reasons for emergence are somewhat obscure; observers believe they abhor strong light. If a filter is rested for several days then placed in use the flies emerge in large numbers, indicating a change of condition is a factor. The greatest risk of nuisance is during a warm night with a very gentle breeze.

Flies which leave the filters live for several days. Those hatched in the laboratory deposited their eggs where there was no organic matter and died soon afterward. It follows that flies coming from the filters have not deposited eggs and, if they are to

perpetuate the race they must return to the filter to deposit eggs, though the author believes sufficient flies remain in the filter to maintain the population.

Remedial Measures.—The author reviews briefly the conclusions of various workers regarding the control of the filter fly nuisance. Removal of the slime from filter surfaces, chlorine, bleaching powder, emulsified creosote oil, ortho-dichlorobenzene, long rest periods for the filters and ammonia are some of the procedures found to have been effective.

The author has used ortho-dichlorobenzene in the form of an emulsion with paraffin oil, which is applied as quickly as possible to the feed to a battery of filters, at the rate of 1 gallon to 200 sq. yds. of filter area. Treatment is begun with the advent of warm weather and continued at 4-day intervals except during cool or wet weather. This treatment, while not completely effective, does obviate the fly nuisance. Lime at the rate of 15 lb. per 100 ft. diameter filter one day each week was not effective.

Discussion.—Mr. H. Taylor (Stalybridge and Dunkingfield) described how he, with the late Mr. Haworth, determined ammonia gas to be effective against filter flies. He was of the opinion that it would be well worth while to try and discover some economical method of getting ammonia gas into the upper layer of the filter and to have it applied at regular intervals where the fly nuisance was acute. The amount of available food, size and type of filter media, and presence of trade wastes appeared to have a considerable effect upon the fly population.

Mr. W. Scott (Bury) described experiments with ortho-dichlorobenzene for filter fly control, using 5 gallons to 980 sq. yds. of filter area. He presented analytical data to show that this chemical had a distinct retarding effect on the purification processes which required more than a week to overcome. Also the effect on the flies had not been very marked and mortality was low.

Mr. J. Sheard (Failsworth) related his experience with bleaching powder for filter fly control. He uses 10 lbs. per filter (60 ft. diameter) each week. The bleaching powder is placed in 40 gallon casks each Saturday and the casks filled with tank effluent. On the Monday following, the whole mixture is well stirred, allowed to settle and the liquid is syphoned off into the chamber feeding each battery of filters.

Mr. T. Barlow (Denton) gave several reasons for freedom from the filter fly nuisance at his works. Lime was used as a precipitant. Then the gelatinous film on the filter medium was controlled by turning over gently, the filter media continuously by means of light rakes attached to the distributor arm. Also a constant supply of *achorutes viaticus* was maintained in each filter. Again, it might be that the fine material in the Denton filter is less favorable to fly breeding than the coarse media generally used.

J. K. Hoskins

FURTHER PURIFICATION BY BACTERIAL FILTERS OF EFFLUENT FROM A PARTIAL TREATMENT ACTIVATED SLUDGE PLANT

By A. S. Miller

Paper presented at the Summer Conference of the Institute of Sewage Purification, Exeter, England, July 2, 1936, and abstracted with discussion in *The Surveyor*, 90, 11–12, 74–77 (July 3 and 17, 1936).

By 1930 the Reading, England, activated sludge sewage treatment plant required a detention period of 18 hours and an air supply of from 2.5 to 3.0 cu. ft. per gallon to produce a satisfactory effluent, with the result that only from one-half to one-third of the dry weather flow could be handled. New sedimentation and balancing tanks were then installed with the expectation that by providing for the removal of the suspended solids from the crude sewage the plant would be capable of treating an increased volume, but this did not prove to be the case. Extending the activated sludge process appeared to be uneconomical under these conditions. It was therefore determined to adapt the plant to partial treatment of the whole flow by the activated sludge process and com-

plete the purification to the required degree on sprinkling filters, if such a procedure could be demonstrated to be practicable. The present paper outlines the experiments undertaken for this purpose.

Nature of Study.—The study involved determination of the reaction of Reading's settled sewage to partial treatment by activated sludge and the possibility of further purification of the effluent on filter beds. The most efficient depth of filter beds, the nature and size of filter medium to be used, and the dosage rate at which the partially treated effluent could be applied to the filters, also had to be considered.

The Thames Conservancy require that an effluent discharging to the River Thames must have: Albuminoid ammonia, less than 2 p.p.m.; or albuminoid nitrogen, less than 1.65 p.p.m.; 5-day B.O.D., not over 20 p.p.m.; and suspended matter, not over 20 p.p.m. At Reading the albuminoid nitrogen requirement was more difficult to meet than that of the 5-day B.O.D. and was therefore used as the criterion of the quality of filter effluents.

Experimental Filters.—Two 25-ft. circular filters, one 6 ft. and the other 10 ft. deep, were constructed with solid concrete side walls and with division walls dividing each bed into three sections. All sections were separately underdrained and filled either with flint gravel, flint gravel topped with Rowley Rag, or destructor clinker, all ½ in. to ¾ in. in size.

Operation.—The filters, dosed with partially treated activated sludge effluent, were put into operation December 21, 1930. After proper seasoning, a continuous feed was applied and then periodically increased. Samples of the effluents were taken every hour and the 24 samples composited to form the daily average sample.

Having determined that the 10 ft. filter was somewhat more efficient than the 6 ft. one, the 6 ft. bed was dismantled, the side walls raised to 14 ft. and the three sections refilled with well washed gravel, one with size ¾ in., one with 1½ in., and the third with 1½ to 2 in.

Various observations, changes in operating procedure, and conditions developing in the different filters during the course of the experiments, are given in some detail.

Conclusions.—A depth of 10 ft. for the filter beds was justified and an application rate of 120 gal. per cu. yd. per day would give a satisfactory effluent throughout the year. Gravel alone as a medium was unsatisfactory, and although topped gravel was favored at first, later results indicated that metallurgical coke was the most suitable medium for the purpose.

Discussion.—Mr. H. C. Whitehead (Birmingham) referred to the pre-treatment of the filter influent and the selection of a suitable filter medium as two of the several ways in which the efficiency of the sprinkling filter might be improved. The work emphasized what Mr. Corbett of Salford had demonstrated 40 years ago—the possibilities of the percolating filter and the importance of preliminary treatment of the influent. Mr. J. H. Kershaw (Rotherham) stated that settled sewage is regularly fed to the bioaeration plant under his charge, trials many years ago having indicated this to be the preferable procedure, and that this method has now become standard practice. His experience with metallurgical coke bore out the conclusions reached by the author. A filter 114 ft. in diameter and 5 ft. deep constructed of this material has been in use at Rotherham since 1905 and is still giving excellent results. He questioned the necessity for deep filters, however. Mr. J. Hurley (Wolverhampton) inquired as to the reason for topping the gravel filter with impervious road-stone or Rowley Rag, a basalt, rather than with clinker or coke. He had topped a filter with 2 ft. of clinker and had obtained very good results. Probably after about nine months the results from the pervious and impervious topping would even up and suggested that if one expected lasting superiority from one kind of medium over another, he might be disappointed. He pointed out that little is known as to why porous materials make better filter media. The supposition that gravel does not support the surface bacterial film is not borne out by the facts. Mr. J. Slater (Chorley) confirmed by his experienee the better results obtained by coke as a filter medium, regardless of whether the filter was fed with tank effluent or activated sludge effluent.

Mr. P. G. Lloyd (Kingston-on-Thames) also had experimented with coke and clinker as filter media, and had found that coke gave the best purification at first but that after 18 months' working the results leveled up and were alike. So long as clinker is cheaper

it will always be preferred. He believed the question of surface area of filters to be extremely important and that depths of over 6 ft. were a step in the wrong direction. Mr. W. H. Makepeace (Stoke-on-Trent) inquired what further change, if any, took place in the oxygen absorbed figure of the liquor in the lower 4 ft. of the 10 ft. filter. Mr. L. F. Mountfort (Edmonton) took issue with the author concerning the superiority of coke over clinker based upon the data presented in the paper. He agreed that generally filters probably deeper than 6 ft. and certainly less than 12 ft. were the more economical. However, the nature of the applied liquid and the method of its distribution were of vital importance in the degree of purification effected by any filter medium. Dr. W. Watson (Burnley), who is also experimenting with the application of final and partially treated effluents from a bioaeration plant to percolating filters, cited some interesting results. He found that the final effluent from the aeration tanks could be treated on percolating filters at the rate of 200 gal. per cu. yd. per day, whereas the partially treated liquor could only be applied at not over 120 gal. to give similar figures for nitrification, etc., although the oxygen absorption was practically the same for both liquors. He inquired as to whether bulking sludge passing to the filter had caused ponding or had affected the quality of the effluent. Mr. E. J. Silcock (Westminster) discussed the importance of the time element in percolating filters and cited two examples—one a very perfect sedimentation and a shallow filter bed of 3 ft. of small grade medium; the other practically no sedimentation and a 12 ft. deep filter of large grade material, the suspended matter being subsequently removed by a sand filter, and in each case the passage time being the same and each producing a satisfactory effluent.

J. K. HOSKINS

JOURNAL AND PROCEEDINGS OF THE INSTITUTE OF SEWAGE PURIFICATION. PART II. 1935

This volume of the *Journal* (pages 231–379) records the activities of the Institute of Sewage Purification for the latter part of 1935 and contains the papers presented before its various meetings during that time. Abstracts of many of these papers have been published in previous issues of the SEWAGE WORKS JOURNAL and are referred to only by title here.

The President of the Institute, Mr. J. H. Garner, reviews, in his usual clear and concise manner (pp. 234–237), the affairs of the organization and then discusses the current outstanding topics concerning sewage treatment and rivers pollution prevention as they have been reflected in the technical press. Aeration processes, although they received wide attention, are not, in the West Riding of Yorkshire drainage area, replacing percolating filter systems. Only ten towns in this region have adopted some type of aeration process in a total of some 430 sewage works of various sizes, and at five of these ten the greater part of the sewage is still treated on percolating filters. Reasons given for this condition are that: (1) percolating filter systems had been constructed before aeration processes had been developed, and (2) the sewage is strong, contains much industrial waste, and towns have therefore hesitated to change from older established practice. Mr. Garner also favors the filtration system from the standpoint of satisfactory effluent, particularly during dry weather.

Sludge digestion plants generally follow German or American practice, but still have unsolved problems of drying, handling and disposing of the digested residue. Satisfactory methods of mechanical dewatering of the digested sludge still need to be evolved.

Treatment of trade wastes and preliminary chemical treatment of sewage to remove abnormalities resulting from contributions of such wastes have been prominent subjects. Legislation is still awaited to regulate the admission of trade waste to sewers and grant statutory powers to local authorities concerning the reception and treatment of such wastes at sewage works.

Centralization of sewage treatment is tending to increase through replacement of smaller units by larger and more efficient plants. Numerous more populous areas, although now supplied with many sewage works, could be benefited by provision of a central scheme that in the long run would prove economical.

Chlorination for odor prevention affords a quick and effective remedy and is becoming increasingly important because of the higher standards being established for the detection, measurement and control of odors.

Description of works of the Wandle Valley Joint Sewage Board. By L. D. Halgate, pp. 240–241.

The sewage from a population of 140,000 is treated in two plants, the South Works dealing with 3 m.g.d. and the North Works 2 m.g.d. of dry weather flow. Purification is effected by means of sedimentation tanks, upward flow Dortmund tanks, rectangular percolating filters and humus tanks. The sludge is filter-pressed with lime. Tank capacities and operating procedures are briefly outlined for the North plant.

Activated sludge research on the practical scale. By W. Watson, pp. 243–261. Abstracted in this JOURNAL May, 1936, pp. 520–523.

Sewage purification and sludge digestion at Coatbridge By A. J. Fulton, pp. 262–282.

The Coatbridge plant is the first one in Scotland for complete treatment of sewage by activated sludge coupled with sludge digestion in heated tanks. It has been in operation for over 2½ years and is designed to treat a dry weather flow of 4.5 m.g.d. and three times this volume in storm times. The present dry weather flow from 43,500 persons is about 3.4 m.g.d. and is rather weak, although it contains such industrial wastes as spent acid from galvanizing works, refuse from acetylene and chemical works, slaughter house refuse and gas house liquors, all of which constitute over 30 per cent of the total sewage volume. The sewerage system is of the combined type.

Screens and Detritus Tank.—A coarse, hand-cleaned screen consists of metal bars set at an angle of 45° and spaced 3 inches apart placed at the inlet end of the grit chamber. This chamber is 45 feet in length and has sloping sides 12 ft. apart at the top and 6 feet at the bottom. At the outlet end is located a "Simplex" grab type screen with metal bars 9 ft. high spaced ½ inch apart, provided with a motor-driven cleaning mechanism.

Pre-sedimentation Tanks.—Six units, providing 8 to 9 hours' detention, are each subdivided to form four compartments, the bottom of each section being an inverted pyramid 22 ft. square. Each tank is 40 ft. deep, a feature of all tanks of the plant necessitated by limitations of the plant site. A sludge well is provided in the center of each unit for extraction of the raw sludge through 4-inch pipes extending almost to the bottom of the tanks. The effluent passes through 3 ft. horizontal slot weirs spaced at regular intervals near the top of the walls, into outlet channels leading to the flow-regulating chamber. The sludge gravitates from the wells to a group of 4 tanks having a combined capacity of 200 tons.

"Simplex" Surface Aeration Plant.—A total of 50 aeration tanks, with a total capacity of 16 hrs. D.W.F., are provided, each 22 ft. square and 23.5 ft. deep, bottoms in the form of inverted pyramids. Each element has a central uptake tube 3 ft. in diameter surmounted by a Simplex aerating cone driven through line shafting by electric motors.

Settling Tanks.—These are in two groups, 8 first and 16 final, with a total capacity of 6 hours D.W.F. All are 22 ft. square, 29 ft. deep and have inverted pyramid bottoms. Activated sludge from the first tanks is returned to the aeration tanks by motor-driven sludge lifters, the volume regulated by valves in the branch pipes. Surplus sludge is pumped back to the pre-sedimentation tanks.

Sludge Digestion Plant.—Three primary and two secondary concrete tanks are each 40 ft. in diameter and 25 ft. deep, built into the ground and earth-insulated. The primary tanks are built with inner and outer walls with a 2-ft. air space between and have floating steel roofs with gas domes. Dorr scum breakers and bottom ploughs on a common vertical, motor-driven shaft move at about three revolutions per hour. Each tank is heated to 70–80° F. by coils of 1½ inch steel pipe suspended around the inner walls and through which water at 130° F. is pumped.

The two secondary tanks are open and unheated, but are equipped with slowly moving bottom scrapers.

Sludge Drying.—About 2¾ acres of drying beds are provided. The medium is broken clinker covered with hard, graded ashes, and is underdrained with tile.

Gas Production.—Gas is produced averaging 0.65 cu. ft. per head per day. The composition is, in per cent by volume: Methane, 65.4; hydrogen, 4.4; carbon dioxide, 30.2.

Of the average 28,365 cu. ft. produced daily, 26.4 per cent is used for heating the coil water, 40.0 per cent is consumed by a 27 H.P. natural gas engine in generating electricity, and 33.6 per cent is burned to waste.

General.—Sludge digestion operations are described in some detail and seven graphs depict various plant operating records over a complete year. An extended discussion amplifies many of the points mentioned in the paper.

Strength of sewage. By M. Lovett and J. H. Garner, pp. 283–307. Abstracted in this JOURNAL, March, 1936, pp. 338–340.

The sewers and works of the West Kent Main Sewerage Board. By A. E. Scott Murray, pp. 308–315.

The Sewerage Board has under its jurisdiction a system serving about 167 square miles, designed for the carriage of sewage and trade waste waters and excluding rain, storm, surface and subsoil waters as far as practicable. Trade waste waters are admitted on payment of charges and compliance with other conditions established by the Board. The flow, about 16 m.g.d., is entirely by gravity to the purification works at Long Reach, Dartford, where the effluent, conforming to the standard of purification required by the River Authority, is discharged into the Thames 5½ miles below one of the main outfalls of the London County Council's main drainage system and at a point where the river is ½ mile wide and 60 ft. deep. The sludge is dried in lagoons and used by farmers for fertilizer.

The treatment works, located along the river front, were constructed during 1922–26 and consist of coarse screen chamber, 2 mechanical screens, 2 detritus tanks of 378,000 gallons capacity, and 24 precipitation tanks of 5.7 m.g.d. capacity. These tanks, 75 x 65 ft. in plan and 7.5 ft. deep, are arranged in six series of four each, through which the sewage, treated with lime and aluminoferric, is passed in continuous flow. The tank floors are pyramidal in shape and equipped with 6-inch outlet pipes for sludge removal. Three storage tanks with a total capacity of 4.4 m.g. are provided to receive the settled effluent during times of high water in the tidal river.

The sludge-drying beds have concrete side walls, a 9-inch layer of ashes on the floors, and are underdrained. Great increases in the sewered population since 1928 have required additional provision for sludge disposal and, after considerable study, three Dorr-Oliver digestion tanks, having a total capacity of 450,000 cu. ft. and with necessary operating equipment, were provided. Each tank is 85 ft. in diameter, 29 ft. deep, is constructed of reinforced concrete and insulated on roof and sides with 2-inch thick slabs of sheet cork. The tanks are provided with top and bottom stirrers and are always kept full, raw sludge being fed in daily at the top and digested sludge drawn from the center of the tank bottom. The digested sludge, after drying to 45 per cent moisture on beds, is pulverized, bagged and sold. Sludge gas, collected in domes on the digester roofs, is stored in a 20,000 cu. ft. gas holder. The gas is composed of carbon dioxide, 25.4 per cent, methane, 66.8 per cent, oxygen, 0.1 per cent, hydrogen, 6.9 per cent, and nitrogen 0.8 per cent, and has a calorific value of 750 B.T.U. per cu. ft. It is used for heating the digesting sludge by means of hot water coils and for generating electric current by a vertical 5-cylinder 250 h.p. gas engine direct-connected to a 172 k.w. generator. About 46,000 cu. ft. of gas per day are used for this latter purpose, so that there is a large surplus for sale or other purposes connected with the Board's undertakings.

Accessory plant and equipment and operating procedures are also described.

The activated sludge systems and percolating filters in the balance. By W. F. Snook, p. 316–321.

The author of this paper discusses the advantages and disadvantages of each of these methods of sewage treatment and suggests that the activated sludge process is in higher favor in the part of England north of Birmingham than in the south, perhaps because standards of purity are not so stringent in the north as in the south. He proposes a State Research Station where work could be carried out on full scale plants in order to speed up progress in solving the problems of sewage purification.

Midland Branch—Visit to Worcestetr

In connection with this meeting the members visited the Worcester sewage works, a description of which was furnished by Mr. Percy Lamb. The 2.5 m.g.d. of sewage is treated by the activated sludge method. One of the four tanks of this plant was used for the first activated sludge scheme in the world, and with slight alterations is still in use, employing the ridge-and-furrow method of aeration.

Sludge is dewatered in lagoons, then pumped to trenches and covered with soil. Because of aerial nuisance, construction has been started on a Dorr clarifier and digester, both of which units are briefly described.

Chlorination in sewage treatment. By R. W. Covill, pp. 324–331.

The author discusses practical points in the use of chlorine in sewage treatment, such as the characteristics of bleaching powder and liquid chlorine and theories of the action of chlorine on polluting organic matter in liquids. Although the primary use of chlorine has been for odor control, it is now also used for increasing the efficiency of filters through control of fly breeding and prevention of ponding; another growing use is for prevention of septicity.

Chlorine demand of sewage is defined as "the parts per million of chlorine as chlorine water (1.0 gm. of available chlorine per liter) required to be added to 250 cc. of the sewage so as to produce a trace (less than 0.1 p.p.m.) of residual chlorine color with O-tolidine reagent after ten minutes' contact of chlorine and sewage." A detailed description is given of a test for estimating the chlorine demand of sewage.

Northeastern Branch—discussion of papers presented at the annual general meeting, pp. 333–342.

Each year at the annual meeting of this branch of the Institute of Sewage Purification papers presented at the annual general meeting of the Institute in London are reported upon by one or two members who had been in attendance and are then discussed by the membership generally. These discussions bring out many interesting views and clarify the subject matter dealt with in the original papers. At this meeting the following papers were considered: "Activated sludge research on the practical scale," by W. Watson (cf. this Journal May, 1936, pp. 520–523), "Sewage purification and sludge digestion at Coatbridge," by A. J. Fulton (see above), and "Strength of sewage," by Messrs. M. Lovett and J. H. Garner (cf. this Journal, March, 1936, pp. 338–340).

Storm overflows. By Thos. Barlow, pp. 342–350.

The general procedure in England is that six times the average dry weather flow of sewage must be passing along a sewer before a storm overflow discharging into a brook or river comes into operation. Furthermore, three times the dry weather flow must receive complete treatment at the works before the excess is permitted to pass direct through special storm water tanks to the receiving water body. The proper design of storm overflows is therefore of first importance to insure the correct diversion of rain and storm water from the treatment plant and trunk sewers. The author describes and illustrates a type of overflow for a works with a dry weather flow of 1 m.g.d. which will effect a division of discharge within 5 per cent of the required amount and which may

readily be varied to cases of other volumes. An important feature of the arrangement is the adoption of a controlling orifice of a given area in which the volume allowed to pass for full treatment varies in proportion to the square root of the head applying, only the velocity being increased.

Factors in the design of sewage disposal works. By H. C. Whitehead and F. R. O'Shaughnessy, pp. 350–371. (Abstract in this JOURNAL January, 1936, pp. 146–147.)

The volume closes with a list of questions given at the Institute's examination, 1935, to establish qualifications of candidates applying for membership in the organization. If the reader desires to test his general knowledge and training in the arts of sewerage and sewage disposal, he is urged to try to answer these questions. He may be surprised to discover his limitations.

J. K. HOSKINS

RESULTS AND EXPERIENCES AT THE BYHOLMEN TREATMENT WORKS AT HELSINGFORS

BY ING. R. GRANQVIST

Gesundh.-Ing., 58, 683, 1935

The Byholmen treatment works consist of a grit chamber with racks and storm overflows, settling tanks with digestion tanks underneath them, secondary digestion tanks activated sludge tanks, secondary settling tanks, sludge drying beds, a machine house and a laboratory building. The sewer system receives storm water as well as sewage.

The ten primary settling tanks, which are modified Imhoff tanks, are arranged around the two secondary settling tanks. The secondary digestion tanks are heated by copper coils placed near the bottom of the tanks. These coils may be removed for examination or repair. One secondary digestion tank, Tank I, is provided with a screw pump similar to those used by Prüss and Blunk and the other, Tank II, is equipped with a simple stirring apparatus. Each tank has a capacity of 10,590 cu. ft. The sludge is pumped from the primary digestion tanks to the secondary tanks and, after complete digestion, flows by gravity to the drying beds. The digestion chambers of the settling tanks are connected in two groups of five tanks each by openings under the sludge surface. The bottom and top of the two secondary digestion tanks are protected against heat losses by a cork layer, 1.2 inches thick in Tank I and 2 inches thick in Tank II. The tops are also made gas tight with lead plates. The supernatant liquor flows back to the digestion chambers of the settling tanks.

The sewage is distributed to the settling tanks through a hopper bottom tank fitted with ten outlets of equal length.

The activated sludge tanks are divided into four groups, each operated in a different way. Group I is operated according to the method of Activated Sludge, Ltd., including reaeration of sludge; Group II differs from Group I in that the walls of the longitudinal ridges are steeper and that there are no division walls; in Group III a longitudinal screen gives the sewage a spiral motion; Group IV is operated with combined compressed air and mechanical aeration. In Groups II, III and IV the sludge is not reaerated.

The Byholmen plant was built for a contributing population of 20,000 with a water consumption of 48 gallons per capita per day. The plant has operated continuously since July, 1932. The population served has increased until it is now about 55,000. In addition, there are industrial wastes from two packing houses, a margarine factory, a silk mill, a dye plant and a large laundry. Although the plant is badly overloaded, the entire dry weather flow is mechanically clarified and a large part of the flow treated biologically. A new secondary sludge digestion tank is under construction.

The bacterial removal averaged 98 to 99 per cent with a maximum of more than 99.9 per cent.

The stage digestion of sludge has been satisfactory although some difficulty has been encountered in the treatment of the sludge liquor. As the water did not separate well from the sludge in the secondary digestion tanks, the supernatant liquor often contained sludge and the tanks into which the liquor was discharged were sometimes overloaded. No appreciable coating of sludge was found on the heating coils. The better insulation of sludge Tank II proved to be well worth the additional cost. At 113° F. the gas yield was about 50 per cent greater than at 77–81° F. but the cost of heating to more than about 81° F. makes it impracticable. The use of stirring apparatus in the digestion tanks at 77–81° F. did not seem to justify the operating cost. The sludge was sufficiently circulated by the gas bubbles and by pumping from the primary settling tanks. At a higher temperature, stirring was necessary to prevent the formation of a hard crust on the heating coils.

The gas produced at the plant was delivered to the municipal gas works. Attempts to dry the sludge on unprepared land were unsuccessful and sludge beds have been used. The freezing of sludge during the winter has been of assistance in drying.

Activated sludge Tank III with the spiral flow gave the best results when effluent, operating costs and area were considered. Sudden changes in the amount or concentration of sewage seemed to affect the purification in this tank less than in the others. The construction cost for this type of tank is somewhat greater than for the other types but this is partially offset by the fact that smaller area and less air are needed. Simplification of design may also reduce construction costs.

The aeration period and the air requirements are largely dependent upon the nature of the waste or sewage to be clarified and care should be taken in comparing the results from plants in different cities. No advantage was observed from reaeration. The dilution of very concentrated sewage by the addition of purified sewage at the inlet to the aeration tank proved beneficial.

A new treatment plant is under construction for the western part of the city.

G. P. EDWARDS

MUNICIPAL SANITATION AND THE DOMESTIC PRODUCTION OF RAW MATERIALS

By Dr.-Ing. Adolf Heilmann

Gesundh.-Ing., **58**, 764, 784, 1935

In spite of the fact that domestic production of benzine in Germany is steadily increasing, during the quarter July–September, 1935, importation of benzine of all types, gas oil and benzol, was 40 per cent greater than in the same period of 1933.

A questionnaire sent to many sewage treatment plants of Germany showed that only at Stuttgart had experimental work been done on the use of methane gas from digestion tanks as a motor fuel. The experiments indicated that compressed methane could be used as motor fuel at a comparatively low cost but this cost was about 40 per cent greater than for a Diesel engine. Now conditions are somewhat more favorable. An Opel passenger car using only two cylinders of gas, each containing 140 cu. ft., can travel about 50 miles in city traffic. Comparison of liquid and gaseous fuel showed that one gallon of liquid is equivalent in heat content to 134 cu. ft. of methane. After light cylinders were developed and after the installation of a new compressor at Stuttgart made possible the use of 200 atmospheres, a five ton truck was operated on methane gas for two months. Five or six light steel cylinders, each of 374 cu. ft. capacity at 200 atmospheres pressure, were sufficient for a day's work. A 2.5 ton street sweeper has also been operated with methane for a month. The hydrogen sulphide is not removed because it aids in detecting escaping gas.

In 1926, motor vehicles were first operated with methane gas, obtained from coke gas. Since these first experiments, methane has been used as a motor fuel to a considerable extent. About 100 vehicles in the Ruhr district are run with this fuel. The

new light weight cylinders of 18.5 gallon capacity, holding 493 cu. ft. of gas at 200 atmospheres, have solved some of the difficulties.

Methane has a heat value of 962 B.T.U. per cu. ft. at 59° F. and 760 mm. and digestion gas, with a composition of methane, 68.55 per cent, carbon dioxide, 29.74 per cent, nitrogen, 1.53 per cent, hydrogen sulphide, 0.1 per cent, oxygen, 0.08 per cent, and a heat value of 732 B.T.U.

If digestion gas is to be used as a motor fuel in competition with other fuels, it must be washed to remove carbon dioxide and compressed. A compressor plant for digestion gas consisted of four stages, had a capacity of 53,250 cu. ft. of raw gas in 16 hrs. and yielded about 35,300 cu. ft. or 72 cylinders of compressed gas. About one third of the raw gas was removed in washing. The compressed gas had a heat value of about 945 B.T.U. The compressor plant was operated by electricity, which was cheaper in this case, and the wash water could be either water or biologically purified sewage.

The cost of the compressor plant was about 37,270 marks and the seamless steel cylinders can be obtained for about 138 marks. These cylinders have a capacity of 493 cu. ft. of gas at 200 atmospheres and weigh 185 lbs. empty. The total cost of 35.3 cu. ft. of gas, purified and compressed, is 0.19 marks at the ditributing point. Trucks usually use a mixture of benzol, benzine and spirits, which costs 1.5 marks per gallon in the Ruhr district or about 5 pf. per 112.4 B.T.U. whereas 112.4 B.T.U. of methane costs 3 pf., a saving of 40 per cent. About 120 cubic feet of purified gas is equivalent to one gallon of liquid fuel.

G. P. EDWARDS

SEWAGE CHLORINATION STUDIES

BY WILLEM RUDOLFS AND HARRY W. GEHM

Bulletin No. 601, 72 pages, March, 1936, New Jersey Agricultural Experiment Station, New Brunswick, New Jersey.

This bulletin deals with some of the fundamental aspects of sewage chlorination as determined by laboratory experiments under controlled conditions. The studies have been divided into four parts, as follows:

I. Chlorine consumption of sewage,
II. Effect of chlorine on flora and fauna.
III. Effect of some physical and chemical factors on the bactericidal action of chlorine,
IV. Effect of chlorine on oxidation.

I. Chlorine Consumption of Sewage.

The gross consumption or chlorine demand of sewage varies considerably and is affected by the type and degree of decomposition and the manner and character of treatment. The chlorine demand of the various components of sewage were studied independently. The soluble components studied included the urine compounds, amino, fatty and hydroxy acids and proteins, soluble soaps and unsaturated compounds; and the suspended materials included the bacteria, the settleable sewage solids and non-settleable sewage solids.

The urine compounds studied included uric acid, creatinin, hippuric acid, urea and ammonium carbonate. Of these the only one with a significant chlorine demand was uric acid, while urea apparently had no chlorine demand. A 10 p.p.m. solution of uric acid had a chlorine demand of 7.6 p.p.m. It was found that while the uric acid content of urine is comparatively low, uric acid is responsible for about 50 per cent of the chlorine demand of urine. The uric acid content of five sewages varied from .2 to 1.5 p.p.m. and was found to be proportional to the chlorine demand of these sewages. Both uric acid and urine react instantly with chlorine as do soluble sewage substances. Urea and uric acid were found to react in the same way with chlorine in sewage as they did in pure culture solutions.

It was found that 4 atoms of chlorine were taken up by one molecule of uric acid and that a carbon linkage was indicated because chlorinated uric acid is not germicidal as are chlorinated amino groups.

For the amino acids tested the chlorine consumption in milligrams per milligram of the compound were as follows: Cystine 1.25, glycocoll, aspartic acid, asparagin, histidine and arginine from .08 to .11 and phenylalinine and glutamic acid 0.0. The figures on the same basis for the soluble proteins were peptone 0.11, albumin .16 and gelatin .08. The fatty and hydroxy acids did not consume chlorine in the dilutions used and can be eliminated as a source of chlorine dissipation. The speed of the chlorine reaction with the amino acids is slightly slower than with the soluble sewage fraction.

Soluble soaps absorb little chlorine though a high rate reaction was indicated for the amount absorbed.

Many unsaturated compounds are formed and destroyed continuously in sewage undergoing decomposition and some of these are present at any time. To determine the reaction of these compounds in high dilution a 1.0 p.p.m. acetylene solution in distilled water was chlorinated. This had a chlorine demand of 5.4 p.p.m. in 5 minutes indicating that acetylene tetrachloride was formed.

Gasoline and kerosene added in amounts of 1 cc. per liter of distilled water showed considerable chlorine demand. It was found that this chlorine demand was largely due to the sulphur content of these materials for when the sulphur was removed about 90 per cent of the chlorine demand was also removed.

The chlorine demands of water suspensions containing 1 billion B. coli per cc. were 6.2 p.p.m. for living and formalin-killed suspensions and 8.0 for heat-killed suspensions. The absorption rate was greatest on living cells. It was calculated that only 0.4 p.p.m. chlorine is required to satisfy the chlorine demand of the 20 mgm. bacterial substance present per liter in normal average sewage and the amount necessary for the living bacteria in sewage was for practical purposes not measurable.

The chlorine demand of the settleable solids portion determined separately was considerable, but the rate of absorption was slower than for any other fraction. When the solids are macerated and dispersed they consumed no more chlorine than the original solids but the rate of absorption was more rapid. An examination of the various compounds found in the settleable solids showed that lignin and the proteins were the only ones active in chlorine absorption. The chlorine demands of the fractions of 5 different sewages were determined and it was found that the settleable solids accounted for 26 per cent of the total chlorine demand, the non-settleable and finely divided 47 per cent, and the soluble 28 per cent.

When fresh sieved or cotton-filtered sewage is chlorinated to satisfy the chlorine demand, from 4.5 to 22.5 per cent of the total suspended solids of the sewage are dispersed. Additional chlorination does not produce any additional dispersion of suspended solids. Septic sewages failed to show appreciable dispersion on addition of chlorine. Chlorination of settleable solids increased the nitrogen content of gooch filtrates. This showed that the material dispersed by chlorine was highly nitrogenous. The nitrogen content of this material is similar to the less soluble amino acids. It was also found that bacterial cells lose nitrogenous materials to the surrounding menstruum upon chlorination.

The residual chlorine determinations were all made using a reagent containing one gram o-tolodine dissolved in an HCl solution containing 180 cc. of concentrated acid per liter. No difficulties were encountered in any of the sewages, effluents, or fractions of sewage used.

II. Effect of Chlorine on Flora and Fauna.

The reduction of the total numbers of bacteria in sewage by chlorination is followed by an enormous rise in numbers or "aftergrowth" on continued incubation. In general, increasing quantities of chlorine increase the peak of the aftergrowth and result in the lengthening of the time required to reach the peak. The reduction of the protozoal enemies of the bacteria by chlorination correlates well with the bacterial numbers reached in the aftergrowth. Chlorine resistant bacteria such as the Chromobacteriae are also an important factor in the aftergrowth. The Coli-aerogenes group shows considerable aftergrowth when sewage is treated with chlorine in quantities up to its chlorine demand.

Experiments with split chlorination showed no consistent increase in the Coli-aerogenes group removal efficiency when the chlorine was applied in a series of doses with 30 second intervals. Split chlorination with two doses to produce a residual chlorine content of 0.02 p.p.m. used an additional amount of more than 7 per cent of chlorine over a single dose. When split chlorination with considerable lapse of time between doses is practiced the effect on bacterial reduction is pronounced due to the action of the chloro-products formed. When chlorinated sewage is added to unchlorinated sewage the same percentage of bacteria are removed by the same final concentration of chlorine in terms of demand as if the chlorine had been added directly.

III. Effect of Physical and Chemical Factors on Bacterial Removal.

The factors studied include temperature, agitation, hydrogen sulphide in sewage, and the effect of the concentration of chlorine added. Experiments were run at 5, 20 and 37° C. and it was concluded that within this temperature range no appreciable differences in the coli-aerogenes reduction or the chlorine demand of fresh or stale sewages occurred upon chlorination. A careful study of the effect of stirring with mechanical stirrers operated from 11 to 210 r.p.m. was carried out. These experiments showed that velocity of agitation has no effect on the immediate kill of the coli-aerogenes group. A single mixing on addition of the chlorine is sufficient to obtain maximum immediate kill. With increasing contact time the percentage of kill is increased both if initial mixing is followed by standing or by continued stirring. Stirring during longer contact periods does not lead to greater kill than does initial mixing alone. The resistance of the organisms and not the lack of contact with the chloro substances explains their survival.

The study of the effect of hydrogen sulphide indicated that it may interfere to some extent with chlorination but that probably other substances present in stale sewage were of greater importance.

The variations in strength of chlorine solution from 100 to 1000 p.p.m. showed no differences in coli-aerogenes kill when the same total amount of chlorine was added to sewage. As no variations of kill occurred by a ten times increasing chlorine solution concentration, it is concluded that the variations in kill due to chlorine concentration would be very small in ordinary practice where the chlorine solution concentration may vary from 300 to 2500 p.p.m.

IV. Effect of Chlorine on Oxidation.

When 5-day B.O.D. determinations are made, a greater reduction in B.O.D. will be obatined if a 10-minute chlorine contact period is allowed than if an additional 4-hour contact period is allowed before diluting and reseeding to make the B.O.D. tests. On longer incubation periods there is little difference in the B.O.D. results for these periods of contact. This indicates that the effect of the chloro-products does not seem to extend beyond the first six days. There is no direct relation between incubation time, B.O.D. reduction and chlorine added. The average 5-day B.O.D. reduction on 42 samples of sewage was 2.4 p.p.m. for each part of chlorine added. The B.O.D. reduction is not constant for each p.p.m. of chlorine added. The parts per million B.O.D. reduction per part per million of chlorine added will depend upon the quantity of chlorine introduced. The B.O.D. reduction is practically constant with quantities of chlorine as low as 20 per cent of the chlorine demand so that the more chlorine added the less apparent B.O.D. is removed per unit of chlorine. This explains the discrepancy in the results obtained in streams or plants where partial chlorination or imperfect mixing is practiced when such results are calculated to a basis of p.p.m. of B.O.D. removed per p.p.m. of chlorine added. This is also correlated with the data in the first section showing that the greatest effect of the chlorine is upon the soluble and semi-soluble material.

Measurements of carbon oxidation indicated that there is retardation in carbon oxidation in chlorinated sewage during the first 5 days. After 10 days the carbon oxidation in the chlorinated sewage was equal to that in the unchlorinated sewage. This study indicated that the chlorine had no oxidizing effect on materials producing CO_2 in sewage and that the differences in the oxidation of these materials upon chlorination was due to a disturbance of the flora and fauna.

A bibliography of 44 references is appended.

C. C. RUCHHOFT

TREATMENT AND RECOVERY OF SULPHITE WASTE

By F. Merryfield

Civil Engineering, 6, 374 (June, 1936)

Civil Engineering presents a three page abstract of an address by Prof. Merryfield of Oregon State College and delivered May 15, 1935 before the Northwest Stream Pollution Conference at Lewiston, Idaho. The article is a summary of efforts that have been made to solve the problem of sulphite wastes, and it suggests lines of further investigation.

Sulphite wastes are a by-product of wood pulp processes. Wood chips are digested under steam pressure with sulphurous acid and calcium acid sulphite. Lignins, polysaccharids and resins are dissolved out and drained off. These with wash water constitute sulphite waste. A typical waste includes 117,000 p.p.m. total solids, practically all in solution, and about 90 per cent combustible. B.O.D. ranges 16,000–25,000 p.p.m. A high initial demand for oxygen is thought to be due to loose combinations of substances and to dissolved gases. Sulphite wastes average perhaps 2,000 gallons per ton and the oxidizable material about one ton per ton of pulp.

More than 1,500 patents have been issued for recovery and utilization processes but none is in general use. Neither is there any present process of treatment that will yield effluents comparable to those from modern swage works.

Prof. Merryfield cites particularly those investigations that have been made, and with quotations abstracted as follows:

Special Rept., Dept. Sanitary Eng., Univ. of Wisconsin, January, 1927: Ponding and aeration of sulphite liquor will reduce the oxygen demand materially, tests showing 76 to 92 per cent in ½-day to 5-day demands, although these results may be high for continued treatment. Aeration, as by Bassley spray, indicate 34 to 60 per cent reduction for 1- to 5-day demands. Ponding avoids intermittent bursts of pollution and, where land is cheap, is nominal in cost. The ultimate solution is to be found in utilization as fuel or for by-product values.

Oregon State College Tests by O'Dell and Greenlaw (*Paper Trade Journal*, August 23, 1934): Under proper conditions, ponding and aeration will reduce pollution effects. Aeration is economically feasible and ponding is useful to tide over low flows in disposal waters.

Review for Sanitary Water Board of Pennsylvania by Baker and Spence, June, 1932: Attempted uses for sulphite liquor have included road or coal binders; gums and sizing materials; tanning materials; alcohol and yeast; fodder; fertilizers. Color is one objection to the first three of these uses; moreover, as binder or sizing, other materials are more adhesive, and for tanning, the market is inadequate. In Scandinavia alcohol was produced as an industry of war time when gasolene was costly. Most of the alcohol plants have been abandoned, although some have turned to yeast. Cattle fodder seems possible and offers a wide market, but the fertilizer market appears best. Nothing has yet been developed for sulphite liquor in either field and possibly because it is too large a problem for any single manufacturer.

Recovery processes now used in the United States include (*a*) concentration by the spray process and (*b*) pressure evaporation both as preliminary to combustion. Plant costs are quoted at $700 to $1,500 per ton capacity. Operation costs are not given. The development of fuel from waste liquors in large volume calls for rapid and costly evaporation of moisture by the use of high pressure steam. This is practicable but the resulting fuel meets local competition from cheap waste wood.

The elimination of pollution from sulphite liquor is a large problem in certain areas. The greater promise of solution appears to lie in methods of utilization, with cattle fodder, fertilizer and fuel as foremost. Pending development of an economical by-product process, treatment by ponding and aeration is effective to some degree, although plant-scale work is necessary before conclusions of efficiency are possible.

Frank Tolles

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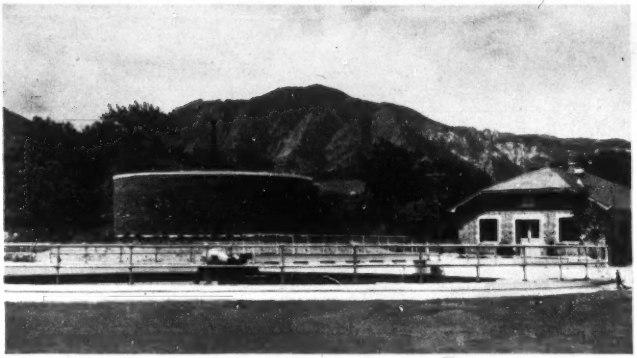

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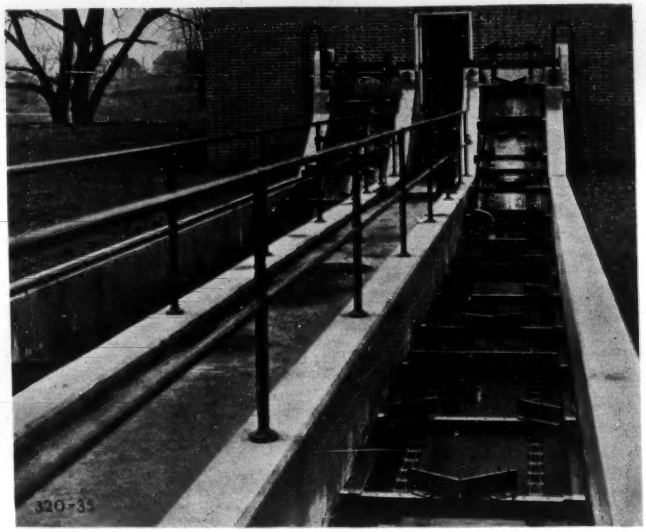

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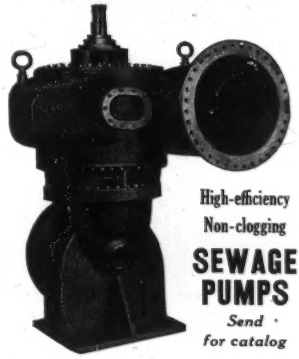

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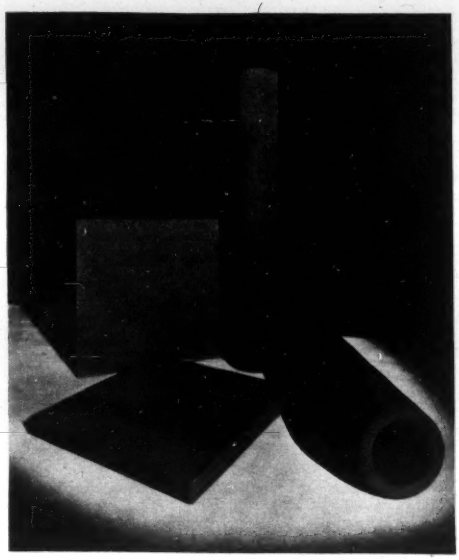

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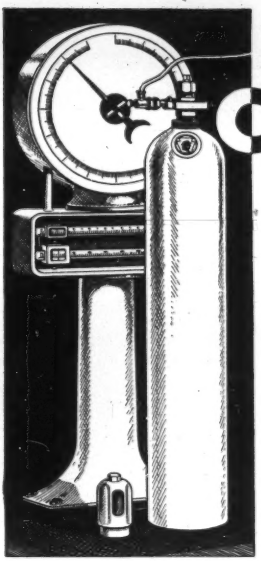

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Georgia Water Works and Sewage Operators' Association
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Pacific Northwest Sewage Works Association
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The Canadian Institute on Sewage and Sanitation
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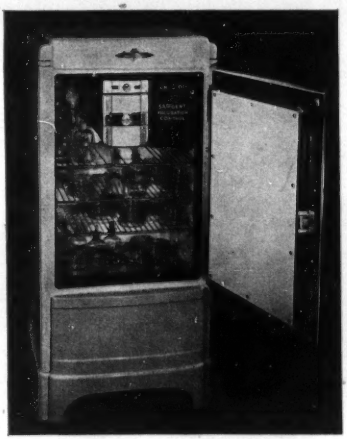

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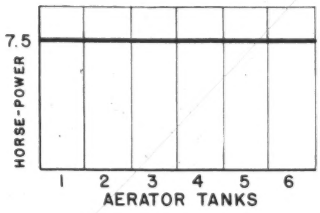
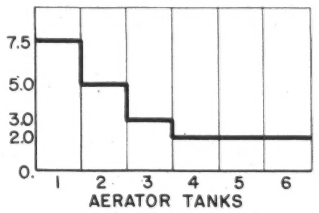


*Fig. 2 — 45 connected horse power would have been required to satisfy oxygen demand by conventional application of mechanical aerators at Monroe.



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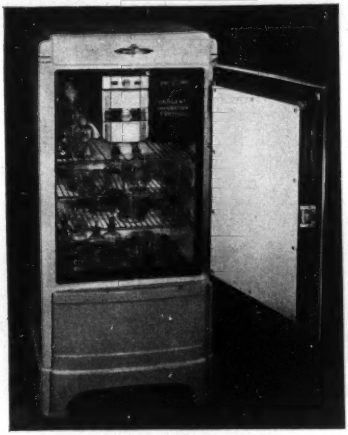

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